Technical Requirements and Geographic Areas for Hydromodification Management in Santa Clara Valley Basin

Final Report

June 2004

by

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Executive Summary

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Hydromodification Management

Santa Clara Valley Basin

INTRODUCTION

On October 17, 2001 the San Francisco Regional Water Quality Control Board issued an amendment to the Santa Clara Urban Runoff Pollution Prevention Program (SCVURPPP) NPDES permit [Order 01-024, February 2001]. Provision C.3 includes a mandate that new development and redevelopment projects must incorporate measures to control the volume, flow, and duration of runoff as necessary to protect streambeds and banks from erosion and other impacts. This provision requires the SCVURPPP to develop a Hydromodification Management Plan (HMP) to govern where and how runoff controls[1] should be implemented.

GeoSyntec Consultants assisted the SCVURPPP in developing a Work Plan, Literature Review, Assessment Method and other tasks required to develop the HMP. The Work Plan submitted to the Regional Water Quality Control Board (RWQCB) was completed in March, 2002. The Literature Review and Conceptual Model for the HMP were developed and submitted to the RWQCB on September 15, 2002. The Literature Review and the Conceptual Model were used to guide the development of the Assessment Method. Technical Memorandum #1, submitted to SCVURPPP, summarized the recommended Assessment Method which was submitted to the RWQCB October 2002.

The Santa Clara Valley Water District (District), as one of the Program co-permittees that would be affected by this provision, hired GeoSyntec to test and validate the assessment methodology. The methodology uses a "*stability index*" that measures changes in flow energy resulting from urbanization. Testing and validating the assessment methodology was necessary to develop confidence that the method could be used to evaluate urban impacts, predict impacts from future development, and evaluate the effectiveness of control measures. The Project Team chose to test and evaluate the methodology on subwatersheds representing distinctly different hydrologic and geomorphic regions within the Basin – the Coyote Creek Watershed, the Guadalupe River Watershed, and the western watersheds.

Three smaller subwatersheds within the larger watersheds were selected to test and validate the method. The initial hypothesis was that these tests would either show that subwatersheds with different hydro-geomorphic characteristics require different management strategies; or

[1] This document focuses on the requirements related to controlling the volume, rate, and duration of runoff. It uses the term runoff controls to refer to Best Management Practices (BMPs) that reduce impacts to runoff volume, rate, and duration. Many runoff controls also remove pollutants from stormwater, but this document focuses on their effect on runoff volume and flow rate.

these tests would show similar results such that a single Basin wide standard and management strategy could be developed. This report addresses this question.

DESCRIPTION OF TEST SUBWATERSHEDS

The initial application of this methodology was to Lower Silver - Thompson Creek, a 42 square mile subwatershed that has undergone significant development and is located within the 320 square mile Coyote Creek Watershed. The study area has experienced rapid development in the last 25 years as agricultural land was converted to residential and commercial properties. During this period, the stream experienced rapid channel enlargement, including stream bed incision and bank failures. No dams or major water diversions are present in this subwatershed. The second test subwatershed was the Ross Creek system, a tributary to the Guadalupe River Watershed, with 170 square mile area. The Ross Creek subwatershed began development in the late 1950's to early 1960's, and thus its observed eroded channel condition has had more time to adjust to hydromodification. The lower portion of Ross Creek in the study area is an engineered earthen channel that shows signs of hydromodification impacts. East Ross Creek, at tributary to Ross Creek is a mostly natural and unmodified creek. No dams or major water diversions are present in this subwatershed. The third subwatershed tested was San Tomas Creek represents the western watersheds. Saratoga Creek is a tributary to San Tomas Creek and the two have a total watershed area of 45 square miles. Saratoga enters San Tomas several miles downstream from the study area. No dams or major water diversions are present in this subwatershed. The results for San Tomas are not complete and not part of the current draft report.

CONTENTS OF THE TECHNICAL REQUIREMENTS REPORT FOR HYDROMODIFICATION MANAGEMENT

This report consists of the following chapters:

Chapter 1 of the report provides summary information regarding the regulatory requirements, goals and objectives of the HMP project, organization of this report which provides two key elements for the HMP (technical requirements and geographic areas for hydromodification) and the definitions of terms used in the report.

Chapter 2 summarizes the overall approach to developing the HMP and its technical requirements. This chapter describes the problem to be addressed by hydromodification management and the relevant physical processes to be considered. It summarizes the assessment methodology developed and tested on the three test subwatersheds and describes the need for a Basin wide standard.

Chapter 3 describes the geomorphic characteristics of the stream systems, the dominant physical processes that seem to be controlling stream erosion, and the extent and modes of failure for observed eroding channel banks and beds. Field crews recorded reach-wide data on streambed and bank erosion, collected data at multiple cross sections within each test

subwatershed, and assigned each cross section a rank of *stable, low, medium or high* observed condition of erosion, which is related to the stability index.

Results:

a) In Thompson Creek, field crews identified six hydrographic segments and 10 geomorphic reaches each with distinct flow and geomorphic characteristics. Data was collected at thirty seven cross sections, although 25 were used in the stability analysis. Stream flow and sediment transport data was collected at four stations within the study area.

b) In Ross Creek, field crews identified six geomorphic reaches based on variation in bed and bank conditions; five along the main stem, with East Ross Creek identified as a distinct reach. Data was collected at 11 cross sections. Hydrographic segments were not defined for Ross Creek due to the high number of stormwater outfalls that discharge to the channel over small intervals of channel length. Clear, well-defined changes in flow volume were not as distinguishable in the channel profile as with Thompson Creek, which had several tributaries and large outfalls that marked clear transitions between different hydrographic segments.

c) In San Tomas Creek, hydrographic segments and geomorphic reaches were not distinguishable due to the high variability within this stream system. San Tomas Creek is fairly diverse in terms of channel form, erosion pattern, and vegetation characteristics compared to Ross Creek. In general, the surveyed portion of San Tomas Creek had less engineered flood control structures than are present in Ross Creek. Data was collected at thirty twelve cross sections.

d) Hydromodification is clearly impacting streams in the Santa Clara Basin. Field observations have found streambed incision from 2 to20 feet depending on stream location. Bank retreat and channel widening is also observed. Eventually banks begin to fail and slough off during larger storms because of over steepened and high banks. Hydromodification impacts are observed for earthen engineered channel as well as natural streams.

Chapter 4 describes the application of the U.S. Army Corps of Engineers' Hydrologic Engineering Center-Hydrologic Modeling System (HEC-HMS) to predict stream flows at selected locations within the study areas. The models were calibrated and verified using two to four years of measured stream flow data. They were run continuously for 50-years to analyze changes in flow duration, runoff volume, and frequency between pre-urban, existing, and future (year 2020) land use conditions.

Results:

a) Urbanization significantly increases the volume, frequency and duration of small to moderate size flows more than those for larger flows. The incremental increase in

work associated with these flows for the post-urban condition exceeds the pre-urban case by as much as 20 to 30 times.

b) In the upper portion of Thompson Creek upstream from Yerba Buena, the 10-year peak flow for the pre-urban condition (~520 cfs) occurs approximately every 5 years under future conditions. Likewise, the 2-year peak flow under pre-urban conditions (~100 cfs) now occurs about once every 15 months. Alternatively, the 10-year peak flow increases from 520 cfs to 730 cfs, (40%) between pre-urban and future conditions. The 2-year peak flow event increased from 100cfs to 270cfs (170%). This represents a dramatic increase in the magnitude and frequency of flows. The increase in flow frequency is less dramatic between existing and future land use conditions, but is still potentially significant.

c) For Ross Creek at its confluence with Guadalupe River, the 10-year pre-urban peak flood event is about 500 cfs. Under existing conditions, that magnitude occurs almost every year. Today the 2-year pre-urban peak of 250 cfs occurs every 1.3 years. The 10-year pre-urban peak flow increased from 500 cfs to about 1500 cfs today. The 2-year peak increased from 250 cfs to about 1100 cfs today.

Chapter 5 presents the stability analysis to evaluate how urbanization changes runoff and the applied shear stresses to streambed and banks. The stability analysis is based on the premise that a balance among flow energy, sediment supply, and channel resilience exists naturally and should be maintained in order for stream systems to remain stable. The stability index represents the "*total effective work*" done on the channel boundary and predicts the likelihood of channel adjustment given watershed and stream hydrologic and geomorphic variables. The index under urbanized conditions is compared to the index under pre-urban conditions, which is expressed as a ratio termed the "*Erosion Potential*" (Ep). The Erosion Potential is then related to the observed stream conditions (stable, low, medium or high). A "*threshold of adjustment*" is discussed in terms of the probability, or risk, that stream channels become unstable due to hydromodification. The Ep and the threshold of adjustment are used to set management criteria and evaluate the effectiveness of management solutions.

Results:

a) Hydromodification and associated channel enlargement must be addressed as a long-term process of cumulative frequent erosive flows. Managing single events, such as the 2-year event, is not protective of streams at risk from hydromodification and should not be used as a management strategy.

b) Matching pre- and post-urban/project flow duration curves is the appropriate management strategy that is protective of stream beneficial uses, including protection from excessive erosion and adjustment.

c) Ep clearly distinguishes between stable reaches (pre-urban, stable/low) and unstable reaches (medium/high) and is an effective tool to address the impacts from

hydromodification.

d) Ep results show a strong indication of a threshold of adjustment. The threshold of adjustment is best expressed as probability or risk of unstable channel conditions using a logistic regression technique.

e) The critical discharge (Qc) upon which bed material begins to move in quantity is about 10 to 15 cfs for Thompson Creek and 20 to 25 cfs for Ross Creek. In both instances, it is 10 percent of the peak 2 year flow.

Chapter 6 discusses the application of the hydromodification assessment method to the entire Santa Clara Basin. Provision C.3.f. requires co-permittees to develop a Basin wide HMP and a single standard is preferred for uniformity and ease of implementation throughout the Basin. The question addressed is "are the Ross Creek, San Tomas Creek and Thompson Creek systems significantly different enough from each other to warrant separate hydromodification standards, criteria and thresholds?" This chapter summarizes the factors used to determine that a Basin wide standard is appropriate.

Results:

a) The Ross and San Tomas Creek systems are not significantly different enough from the Thompson Creek system to warrant separate hydromodification standards, criteria or threshold of adjustment in different parts of Santa Clara Basin.

b) For the test subwatersheds, the transition between stable and unstable channels occurs between Ep ratios of 1 and 2.

c) The estimated *Range of Storms* to manage (Qc to the 10-year peak flow) is consistent between subwatersheds. Specifying Qc for development as "*10% of the 2-year pre-project peak flow*" is consistent between subwatersheds.

Chapter 7 summarizes the approach to determine geographic areas within Santa Clara Basin that can be exempted from the HMP requirements. Provision C.3.ii allows an exemption for discharges into creeks or storm drains where the potential for erosion, or other cumulative impacts, in highly developed watersheds is minimal. A method that accounts for the cumulative effects of development at points along the stream system are described in this chapter. The method includes current measurements of unstable stream channel conditions and the magnitude of future development above current levels that will destabilize additional stream segments that are not hardened.

Results:

a) Provision C.3.f.ii allows for exempting discharges from certain developments where the potential for erosion and other cumulative impacts is minimal. The Project Team developed two supporting methodologies that delineate exempt areas within the Santa Clara Basin.

i. The first part of the method determines the most downstream limit (MDL) of erosion in each watershed, upstream of which hydromodification controls are required. The MDL defines the point where current active erosion is observed and where any further increase in watershed development will exacerbate the observed erosion problems. Developments that discharge stormwater above this point cannot be exempted from the HMP unless other measures are taken to mitigate the expected impacts.

ii. Downstream from the MDL and in stable segments between MDLs, it becomes less certain that future development will, or will not, increase the risk of erosion. The second part of the method considers the increases in percent imperviousness in the future above existing levels at which the currently stable segments may destabilize in the future. This approach identifies a point in the stream system where the increase in percent impervious surface area from future development is "minimal". The Project Team recommends that a 3% change in percent imperviousness from existing to future conditions (measured on the total area of the watershed) be set as the limit to before which additional studies for geomorphic stability are required

iii. *It should be clear that the method does not rely on any assumed value of future imperviousness.*

Chapter 8 provides a discussion of various issues regarding management strategies and implementation requirements. Several management strategies have been tested for their effectiveness at protecting the beneficial uses of streams. This chapter summarizes the relevant topics and conclusions from these tests. Appendix B of the report provides copies of the relevant technical memorandums summarizing these tests.

Results:

a) It was originally thought that practical implementation could be achieved by allowing smaller developments to perform simpler calculations, such as discrete event volume control. However, after further study, simpler methods did not prove to be adequate to protect stream channels from hydromodification because of the cumulative nature of smaller but erosive flows.

b) Flow duration control was found to be effective at maintaining the erosion potential of the stream. Flow duration control is more accurate for sizing controls because it maintains runoff volume for the full distribution of flows. Flow duration control matches the total number of hours that flows persist at any given magnitude.

c) With Ep values above 0.8 there is some risk of still finding unstable channel conditions. This risk is associated with uncertainties and variability with the analytical method and with the data collected in the field. At an Ep of 1.0, there is an 8% (1 in 12) risk that stable stream segments will become unstable.

d) Based on the technical analysis, it is possible for the District to select an acceptable risk of unstable channel conditions of 5 percent (risk of 1 in 20 streams becoming unstable). This is observed at an Ep of 0.8. When the out-of-stream HMP facilities discharge at an Ep > 0.8, the Ep can be reduced to 0.8 by using a set of in-stream measures. In-stream measures such as grade controls, cross section geometric changes, flood plain creation, bank protection, and bypass of flows from upstream areas can reduce the Ep achieved with out-of-stream measures.

e) In some cases, there may be space limitations, soil infiltration limitations, or other design and construction limitations that will prevent a developer from matching flow duration curves. When the developer is not able to match the pre-project flow duration curve, the developer should work with the District to find a means to mitigate the residual work.

f) Since the value of Ep in the channel is based on all of the cumulative changes that take place in the watershed, consideration must be given to changes in runoff that are not subject to HMP requirements. For example, compaction and impervious area outside the development subject to HMP requirements (such as service roads to the development). Risks that are not captured in on-site or off-site controls have to be managed through other measures, such as in-stream upgrades.

The technical and geographic requirements for the HMP were prepared in response to changes in the Regional Water Quality Control Board (RWQCB) National Pollutant Discharge Elimination System Provision (NPDES) C.3 permit requirements. RWQCB Order 01-119 [October 2001] amended Provision C.3 of the SCVURPPP NPDES permit [Order 01-024, February 2001] to incorporate hydromodification management requirements. In addition to complying with the RWQCB order, the assessment method has application as a planning and design tool for future stream rehabilitation and flood management projects. The method is capable of evaluating the long-term stability of earthen engineered channels.

The methodology can be used to address cumulative watershed scale modifications during master planning or for larger scale developments, such as in the Evergreen Specific Plan in the Thompson Lower Silver Watershed. Ep can be used to evaluate the effectiveness of management strategies, including on-site, regional, in-stream projects, or a mix of strategies.

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1 Introduction

1.1 About this Document

The technical requirements and geographic areas for hydromodification management in Santa Clara Basin have been prepared in response to the Regional Water Quality Control Board (RWQCB) National Pollutant Discharge Elimination System Provision (NPDES) C.3 permit requirements to develop a Hydromodification Management Plan (HMP) .

The RWQCB Order 01-119 [October 2001] amended Provision C.3 of the Santa Clara Urban Runoff Pollution Prevention Program (SCVURPPP) NPDES permit [Order 01-024, February 2001] to incorporate hydromodification management requirements. Provision C.3 includes a mandate that new development and redevelopment projects must incorporate measures to control the volume, flow, and duration of runoff as necessary to protect streambeds and banks from erosion and other impacts. This provision requires the SCVURPPP to develop a HMP to govern where and how runoff controls[1] should be implemented.

The GeoSyntec Project Team began by assisting the SCVURPPP in developing a Work Plan, Literature Review, Assessment Method and other tasks required to develop the final HMP. The Work Plan submitted to the Regional Water Quality Control Board (RWQCB) was completed in March, 2002. The Literature Review and Conceptual Model for the HMP were developed and submitted to the RWQCB on September 15, 2002. The goals and objectives in the Literature Review and the Conceptual Model were used to guide the development of the assessment method. A technical memorandum (Technical Memorandum #1) summarizing the Team's recommended assessment method which was submitted to the RWQCB October 2002.

The GeoSyntec Project Team was then hired by the Santa Clara Valley Water District (SCVWD) to test and validate the assessment methodology. This implementation involved testing the method in terms of predicting erosion caused by urbanization, and evaluating the effectiveness of control measures.

Testing the methodology involved applying the method to three distinct subwatersheds within the Santa Clara Valley (Basin). The C.3 provision requires a Basin wide HMP. The Project Team chose to test and evaluate the methodology on three subwatersheds representing the three distinctly different hydrologic and geomorphic regions within the Basin. The initial hypothesis was that these tests would either show that subwatersheds with different hydro-geomorphic characteristics require different hydromodification standards and management strategies; or these tests would show similar results such that a single Basin wide standard and management strategy could be developed. This report summarizes the results addressing this question.

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[1] This document focuses on the requirements related to controlling the volume, rate, and duration of runoff. It uses the term runoff controls to refer to Best Management Practices (BMPs) that reduce impacts to runoff volume, rate, and duration. Many runoff controls also remove pollutants from stormwater, but this document focuses on their effect on runoff volume and flow rate.

1.2 Regulatory Framework

The California Regional Water Quality Control Board (RWQCB) is the regulatory agency that issues National Pollutant Discharge Elimination System (NPDES) permits to municipal agencies for stormwater discharges to San Francisco Bay. Beginning in 2000, in response to direction from the State Water Resources Control Board and changes to NPDES permits in Southern California, the RWQCB began to reissue or amend existing permits to include stricter requirements for control of stormwater from new development and redevelopment. The first Bay Area permit to include the new requirements was that of the Santa Clara Valley Urban Runoff Pollution Prevention Program (SCVURPPP), which is comprised of the thirteen cities of Santa Clara County, Santa Clara County, and the Santa Clara Valley Water District (SCVWD), all of which are Co-permittees, also referred to as Dischargers. (The RWQCB's Order 01-119 [October 2001] amended Provision C.3. of SCVURPPP's reissued NPDES permit [Order 01-024, February 2001] to incorporate the requirements.)

Provision C.3. of the NPDES permit mandates that the Co-permittees require development and redevelopment projects above certain impervious surface thresholds to incorporate measures to improve the quality of stormwater runoff. Under Provision C.3.f., *Limitation on Increase of Peak Stormwater Runoff Discharge Rates*, certain new development and redevelopment projects must also control the volume, peak flow, and duration of runoff as necessary to protect streambeds and banks from erosion and other impacts. That provision requires the Co-permittees to develop a Hydromodification Management Plan (HMP) to govern where and how runoff controls should be implemented.

In summary, Provision C.3.f. requires runoff controls for all new development and redevelopment projects that:

- Meet the impervious surface area criteria in provision C.3.c.; *and*
- Would produce runoff that (if not controlled) would increase erosion in or otherwise harm beneficial uses of receiving streams (see Provision C.3.f.i).

When needed, runoff controls must be designed so that "post-project runoff shall not exceed estimated pre-project rates and/or durations" (from Provision C.3.f.i.). However, on-site runoff controls can be reduced or eliminated under several conditions. Specifically:

- Provisions C.3.f.ii and C.3.f.iii eliminate or reduce runoff controls for projects that discharge stormwater runoff where the potential for erosion, or other impacts to beneficial uses, is minimal. Such situations may include discharges into creeks that are concrete-lined or significantly hardened (e.g., with rip-rap, sackcrete, etc.) *downstream to their outfall in San Francisco Bay*, underground storm drains discharging to the Bay, and construction of infill projects in highly developed watersheds, where the potential for single-project and/or *cumulative* impacts is minimal.

Provision C.3.f.vii allows for addressing flow impacts through the development of a protocol for using strategies other than on-site runoff controls (such as in-stream control measures).

The protocol may allow increases in volume and/or durations, subject to the implementation of in-stream measures (restoration in advance of development generated flows) that would accommodate expected additional flows without harming beneficial uses.

Provision C.3.f.vi indicates that the HMP may include:

- Evaluation of the cumulative impacts of urbanization of a watershed on stormwater discharge and stream morphology in the watershed
- Evaluation of stream form and condition, including slope, discharge, vegetation, underlying geology, and other information as appropriate
- Implementation of measures to minimize impervious surfaces and directly connected impervious area in new development and redevelopment projects
- Implementation of measures including stormwater detention, retention, and infiltration
- Implementation of land use planning measures to allow expected changes in stream channel cross sections, stream vegetation, and discharge rates, velocities, and/or duration without adverse impacts to stream beneficial uses
- A mechanism for pre- vs. post-project assessment to determine the effectiveness of the HMP and to allow amendment of the HMP, as appropriate
- Other measures, as appropriate.

1.3 Goals and Objectives

The primary goal of the HMP is to protect the physical, chemical, and biological functions of urban stream systems, subject to considerations of benefits and costs. Benefit-cost considerations suggest two secondary goals based on stream conditions. A top priority is protection of existing healthy stream systems with a goal that future urbanization will not result in a net loss of ecological functionality. For streams that are eroding due to urbanization, the goal is to not exacerbate the observed erosion problems, and meet water quality needs.

The emphasis of the HMP is on understanding and managing the cumulative effects of urbanization on hydrology, water quality, and geomorphology on a watershed scale. Controls for the watershed will be prescribed to address future risks to downstream levees, and creek banks and beds caused by the cumulative impacts from urban development.

In order to meet the above goals and the requirements of Provision C.3., the following objectives were defined:

- Develop a Basin wide HMP for approval by the RWQCB to address the impacts of hydromodification on the beneficial uses of streams.

- Develop, test, and apply an assessment method to evaluate the cumulative impacts from hydromodification.

- Develop design criteria, control measures, and guidance on management strategies to address hydromodification and identified impacts.

- Manage the impacts of hydromodification on streams through the implementation of the HMP.

- Monitor the effectiveness of the control measures and management strategies, and amend the HMP as needed.

1.4 Organization of the Report

This technical report, which documents the technical and geographic requirements for a basin-wide HMP in Santa Clara Basin, is organized into the following eight chapters:

- Chapter 1 Introduction

- Chapter 2 Background

- Chapter 3 Geomorphic Assessment

- Chapter 4 Hydrologic and Hydraulic Modeling

- Chapter 5 Stability Assessment

- Chapter 6 Application Methodology to All Watersheds

- Chapter 7 Exempt Areas Analysis

- Chapter 8 Method Limitations and Uncertainties

1.5 Terminology Used in This Report

A *watershed* is the land area that drains to a given water body or watercourse. Any given watershed is typically part of a larger watershed (associated with a larger water body or watercourse) and, in turn, is typically comprised of smaller ones (associated with smaller, tributary watercourses). This document uses the following terminology to distinguish watershed levels:

- The Santa Clara Basin (or the Basin) refers to the 840 square miles that drain into San Francisco Bay south of the Dumbarton Bridge.

- The major streams within the Santa Clara Basin—such as Guadalupe River—define watersheds, which are named for the associated watercourse (for example, the Guadalupe River Watershed).

- The term "subwatershed" refers to the land that drains to a tributary of one of the major streams (for example, the Ross Creek subwatershed within the Guadalupe River Watershed).

- The term "catchment" refers to an area that drains to a single storm drain outfall.

- Hydromodification is the term used to refer to changes in runoff characteristics and associated stream impacts that result from land use changes. Several factors and processes interact to influence hydromodification.

2 Methodology

2.1 Problem Statement

Urbanization of a watershed can profoundly change the physical characteristics of streams, harming stream habitat and beneficial uses. Urbanization is defined as the transformation of open undeveloped land into residential, commercial, and industrial properties and associated infrastructure such as drainages, roads, and sewers. Urbanization modifies vegetation and soil characteristics, introduces pavement and buildings (impervious surfaces), and creates drainage and flood control infrastructure.

As the area of impervious surfaces increases, infiltration of rainfall decreases, causing more rainfall to run off the surface as overland flow (stormwater runoff), and decreasing the time between when rainfall occurs and when runoff occurs. Since runoff ultimately discharges into streams (and other water bodies), increases in the frequency of events, volume and rate of runoff increase the duration of erosive stream flows. This effect is more pronounced for more frequent smaller storms than for the less frequent larger storms responsible for flooding.

Increases in the frequency and duration of erosive stream flows intensify sediment transport, causing excessive erosion and modifying the geomorphology (width, depth, and slope) of stream channels. These effects also impair the habitat in stream channels. These types of changes have been documented in numerous streams throughout the Santa Clara Basin, including the Lower Silver - Thompson Creek and the Ross Creek subwatersheds.

Although the HMP focuses on the effects of urbanization, streams may also be impacted by historical land use practices such as grazing, agriculture, and mining; the development of infrastructure such as bridges, flood control and water supply facilities; and buildings located in the floodplain. Natural events such as fires, droughts, and seismic activity can also impact stream geomorphic and habitat characteristics. Potential stream impacts include streambed incision; bank failures; siltation; temperature increases; water quality degradation; and loss of habitat features such as suitable substrate, pools, tree cover, and overhanging banks. The test subwatersheds selected for the development of the HMP have primarily urban influences with little other historical impacts.

2.2 Overall Approach to Developing the HMP Requirements

Developing the HMP Requirements included the following tasks:

- Develop a problem statement and goals.
- Review literature and data availability.
- Convene an expert panel for continuous review
- Characterize stream conditions and future development patterns.

- Develop a conceptual model of the hydrologic and geomorphic processes relevant to hydromodification.
- Develop a method and model for computing stream stability.
- Compile available data and apply the model to determine stream impacts.
- Evaluate and recommend cost effective solutions to address the impact on streams.
- Develop guidance for hydromodification control measures.

The expert panel has assisted the project team throughout the study. The goals and objectives and conceptual model guided the development of an overall framework for conducting the assessment.

2.3 Conceptualization of Relevant Watershed Processes

Figure 2-1 illustrates a conceptual model of the hydrologic and geomorphic processes relevant to hydromodification. Regional factors of climate, geology, and physiography affect the amount of runoff and sediment discharged to stream channels. Land use, soil, and vegetation characteristics affect the proportion of rainfall that infiltrates into the ground or runs off the surface. Local climate, geology, and physiography also affect the type and amount of sediment that is supplied to the stream system. The changes in stream flow and sediment load ultimately change the physical and ecological characteristics of the stream channel.

Urbanization causes immediate changes in stream flow, but it takes longer for the stream channel morphology to change in response. The time it takes depends on many factors including geology, soils, topography, vegetation, and the extent of development in the watershed. Therefore, the time varies among different watersheds, and even within the same watershed, different geomorphic reaches will respond differently to the same change in stream flow. Stream channel changes due to hydromodification generally occur within a decade or so after development occurs. However, it is possible to observe the effects of hydromodification within the first winter season in streams with a high susceptibility to erosion or if a large storm occurs soon after development. Conversely, stream channel response to hydromodification may take longer than a decade, if the channel and/or bank material is very resistant to erosion (e.g. hard clay or bedrock).

2.4 Overview of the Assessment Methodology

The assessment methodology was subdivided into five elements: 1) watershed and problem characterization, 2) geomorphic assessment, 3) hydrologic modeling, 4) stability assessment, and 5) evaluation of effectiveness of solutions.

Problem area and reach characterization describes features of the watershed and stream channels necessary to understand the nature and extent of the problem and to explain existing conditions. Historical aerial photography, soils and geologic maps, channel maintenance records,

infrastructure data and historical surveys were used to distinguish between urbanizing impacts and impacts caused by past land use practices.

The geomorphic assessment describes the geologic and geomorphic characteristics of the stream network, the dominant physical processes that seem to be controlling stream attributes and erosion, and the extent and modes of failure for observed eroding channel banks and beds. Field crews recorded reach-wide observations on streambed and bank erosion, collected location specific data at multiple cross sections, and then assigned each cross section a rank of *stable, low, medium or high* observed condition of erosion, or likelihood of erosion in the near future. Several factors affecting channel bank and bed stability were combined with the extent, age, and magnitude of existing erosion to designate an appropriate erosion ranking.

A hydrologic continuous simulation model of the pre-urban and existing land use conditions was developed using the U.S. Army Corps of Engineers' Hydrologic Engineering Center - Hydrologic Modeling System (HEC-HMS). The models were calibrated using two to four years of measured flow data from pre-urban and existing periods, or from existing data from undeveloped and developed portions of the watershed. The calibrated models were run using 50+ years of rainfall records, which produced a time series of hourly stream flows to analyze changes in flow duration, runoff volume, and frequency between land use scenarios.

The stability assessment is based on the premise that a balance among flow energy, sediment supply, and channel resilience exists naturally and should be maintained in order for the stream network to remain stable (GeoSyntec, 2002; McRae, 1996). The hypothesis is that, over time, the stream channel slope and geometry co-evolved with vegetation, local physiography and climate to establish its pre-development dynamic equilibrium. To maintain stream channel stability, the management goal is to maintain the pre-development flow energy, sediment transport and erosion characteristics.

An *index* representing the *total effective work* done on the channel boundary is derived that predicts the probability of channel adjustment given watershed and stream hydrologic and geomorphic variables. Figure 2-2 provides an illustration of the effective work index and its definition related to the stream cross section. The effective work index (W) is computed as the excess shear stress that exceeds a critical value for streambed mobility or bank material erosion integrated over time and represents the total work done on the channel boundary. Other terms in the equation include C = constants and unit conversions, τ_c = critical shear stress that initiates bed mobility or erodes the weakest bank layer, τ_i = applied hydraulic shear stress, Δt = duration of flows (in hours), and n = length of flow record (number of hours).

The effective work index under urbanized conditions is compared to the index under pre-urban conditions expressed as a ratio (Ep). Figure 2-3 provides an illustration of the Ep ratio and its definition related to the time series of flow and excess shear stress. The comparison, expressed as a ratio, is defined as the Erosion Potential (Ep) following McRae (1992, 1996); where W_{post} = work index estimated for the post-urban condition and W_{pre} = work index estimated for the pre-urban condition.

An empirical relationship between Ep and the observed erosion classification is derived and used to make informed decisions. A "threshold of adjustment" is considered that distinguishes

between stable and unstable conditions. The Ep and the threshold of adjustment are used to set management criteria and evaluate the effectiveness of proposed solutions (McRae, 1993).

2.5 Development of a Basin Wide HMP Requirements

Provision C.3.f. of the NPDES permit requires Co-permittees to develop an HMP that applies to the entire Santa Clara Basin under the jurisdiction of the SCVURPPP.

Ideally, the Santa Clara Basin would be represented by a single standard, management criteria and thresholds; for uniformity and ease of implementation.

However, according to geomorphic theory, watersheds and streams with different climate, physiography, soils and vegetation, could potentially have different thresholds of adjustment and thus require different standards. On the other hand, if the assessment methodology is parameterized effectively incorporating the relevant physical processes, the method could account for these differences and lead to uniform results.

The Project Team analyzed the Santa Clara Basin and concluded that it could be sub-divided into three regions with similar characteristics: the Coyote Creek watershed, the Guadalupe River watershed, and western watersheds. The Lower Silver – Thompson Creek subwatershed was selected to represent the eastern Coyote Creek region. The Ross Creek – East Ross Creek subwatershed was selected to represent the Guadalupe River region. The upper portion San Tomas Aquino Creek subwatershed above Williams Road was chosen to represents the characteristics of the western watersheds. Figure 2-4 presents a map showing the Basin and the location of the three test subwatersheds.

The first test subwatershed was used to develop, test and verify the assessment method itself and ask broader questions of adequacy. For example, "can one correctly predict existing erosion and deposition?" "How successfully can one define erosion thresholds?" The second and third test subwatersheds were used to verify that the method works in a watershed with different characteristics; and ask two fundamental questions. These two questions are:

- Are the Ross Creek and San Tomas Creek results significantly different enough from Thompson Creek to warrant a separate hydromodification standard, criteria and threshold? or
- Are the results similar enough such that a single Basin wide standard, criteria and threshold can be developed?

The subwatersheds were selected for the following reasons:

- Each subwatershed represents a distinct hydro-geomorphic region of the Basin.
- The subwatersheds are of a size appropriate for the testing methodology (approximately 20 to 40 square miles).

- Each subwatershed has a stream gage(s) whose data can be used to help calibrate the hydrologic model.

- The subwatersheds have undergone significant development in the past 20 to 40 years. Thompson Creek underwent significant development from the late 1970's, while development in Ross and Upper San Tomas Creek began around the late 1950's.

- The streams are non-hardened and show signs of hydromodification type impacts due to urbanization. Other effects, such as dams and reservoirs, gravel mining were not present in these watersheds.

3 Field Geomorphic Assessment

3.1 Overview

A field geomorphic assessment was completed for the subwatersheds of Thompson, Ross and San Tomas Creeks as part of development of Hydromodification Management requirements. Conducting a geomorphic assessment is a critical step toward understanding the hydrologic and geomorphic processes that influence channel stability within an urbanizing watershed. The historic assessment and geomorphic field work conducted for this study provide the foundation for evaluating how past hydrology[2] has changed, how modified flow pathways have affected channel form and stability, and what this information means in terms of how the system might be managed to create an ultimately stable channel.

The geomorphic assessment complements and supplements information addressed by hydrologic models of storm runoff and stability assessment, and it helps in the interpretation of what the models mean in different stream systems.

This chapter describes:

- Previous work that guided the creation of a geomorphic assessment program
- The scales at which the geomorphic assessment was conducted (the identification of geomorphic reaches and hydrographic segments)
- Methods used to conduct the geomorphic assessment
- Results and discussion of the field surveys
- Subwatershed comparison

3.2 Previous Work

Several studies and reference items guided the geomorphic assessment and provided a strong framework for setting up and interpreting the results of the field survey:

- Santa Clara Valley Water District's Yellow Book[3]

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[2] In this usage, hydrology is taken to be the flow conditions affecting channel stability, with emphasis on the magnitude and duration of storm-flow events, but also including consideration of sediment and large-wood transport from upstream, channel-disturbing episodic events, and the droughts which may have affected riparian vegetation sufficiently to be geomorphically significant.

[3] The Santa Clara Valley Water District Maps of Flood Control Facilities and Limits of 1% Flooding (also known as the "Yellow Book") was prepared in June of 1993 by the Flood Control Planning Division and the Engineering Services Drafting Section. It contains maps of all of the creeks within the Santa Clara Valley Basin at a scale of 1"=1000'.

- Information from the SCVWD maintenance crew on stream conditions, such as the location and extent of bank erosion

- Design drawings for Ross Creek channel modification from 1955 and 1957 generated by the Santa Clara County Flood Control and Water Conservation District

- Historical USGS topographic maps:

    a. San Jose quadrangle – 1899, 1943, 1953, 1961

    b. Mt. Hamilton quadrangle – 1897, 1943, 1947

    c. Palo Alto quadrangle – 1899, 1961

    d. Los Gatos quadrangle – 1919, 1940

A consistent set of historical aerial photographs was sought, but not found, during this study.

3.3 Divisions Used in the Assessment

Creeks vary longitudinally—that is, from upstream to downstream, both with respect to channel bed and bank properties and with respect to the magnitude and duration of flows which act on them. Both the intrinsic properties of channel bed and banks plus the flows acting on them need to be considered when developing a method for predicting how future flows will affect the creek. Therefore, two different channel divisions were utilized for this study – geomorphic reaches and hydrographic segments, which explore the intrinsic properties of channel bed and banks and flows acting on them respectively.

3.3.1 Geomorphic Reaches

The project team distinguished subwatershed-specific *geomorphic reaches* that characterize each creek based on the longitudinal variation of channel bed and bank conditions. Within a given geomorphic reach, broadly-similar influences (whether vegetation, geology, topography, level of upstream development, etc. or some combination therein) affect channel form and processes in a similar manner. Sometimes, it is useful to recognize sub-reaches within a given reach.

The geomorphic reaches developed for the studied watersheds should not be interpreted as a regional classification system, but rather they were developed for description purposes aimed specifically for the development of the hydromodification management requirements. The identification of geomorphic reaches allows some generalization of channel characteristics and easier identification of patterns within specific sub-watersheds.

3.3.2 Hydrographic Segments

Hydrographic segments differ from geomorphic reaches in that they define a creek section based on significant flow additions, whether it is from tributaries, diversion structures, and/or stormwater outfalls, rather than physical characteristics of the channel. Cross-sections located

within a common hydrographic segment are characterized as having similar discharges. The segmentation reflects important differences in channel-forming flows.

Hydrographic segments are more fully explored and utilized in Chapters 4 and 5.

3.4 Work Conducted

The geomorphic field work consisted of taking a set of qualitative and quantitative measurements at multiple cross-sections along Thompson, Ross, and San Tomas Creeks. Data collected at these cross-sections were primarily used as input parameters for the stability assessment calculations (discussed in Chapter 5), but were also used to interpret and evaluate feasibility of the results of the stability assessment and provide a field-based understanding of the studied subwatersheds.

Cross-sections were selected in each subwatershed to represent both typical "problem sites" and "stable" sections of creek that are not experiencing excessive and/or accelerated erosion. The selected problem sites typically reflected larger-scale erosion patterns versus site-specific erosion.

One of the goals of this study is to evaluate and test the HMP assessment method. Selecting cross-sections at sites that are currently eroding and stable allows us to test if the stability assessment model can differentiate between the two conditions.

3.4.1 Field reconnaissance

The project team conducted field reconnaissance surveys along the studied creeks during 2002 and 2003. The site visits consisted of walking most of the length of the creeks and noting problem areas, zones of deposition, sediment transportation and erosion, in-stream and bankside structures, and other qualitative observations on the geomorphic characteristics of the streams. Also as part of the field reconnaissance, the project team measured depth from "hanging" structures (such as pipes and outfalls) to the existing channel bed to obtain a rough estimate of how much incision has occurred at a specific site. The hanging structures were dated where possible to give a maximum period over which incision may have occurred.

3.4.2 Cross-sectional channel geometry

Channel geometry is used in the stability assessment model to calculate water depth, velocity and other hydraulic parameters. The project team measured channel geometry at each problem or non-problem ('stable') site by establishing a cross section that best represents the site. Each cross-section was monumented using a piece of rebar to allow future re-surveying and comparison.

The project team then measured the following:

- Cross-section distance (or 'station') across the channel (using a tape)
- Relative elevations of points along the cross-sections (measured using a automatic level and stadia rod) —see Figure T3-4

- All major breaks in slope, with particular focus on top of bank, toe of slope, edge of channel and in-channel bars

3.4.3 Longitudinal profile

A longitudinal survey of Ross Creek was performed by William Lettis & Associates, Inc.[4], on August 4, 5, 7; September 16, 17; and October 16 of 2003.

A TopCon digital laser theodolite total station was used to collect the Northing (N), Easting (E), and Elevation (Z) data for the longitudinal survey. The instrument was oriented to north before each survey using an azimuth estimated by Brunton compass. Survey data was first collected in an arbitrary Cartesian coordinate system, and later tied in to SCVWD benchmark coordinates.

Surveyed data were translated into State Plane coordinate system (Zone III, NAD 83). This was done by calculating the linear difference between the benchmark x, y, and z collected from our survey and the x, y, and z provided by the District. In a spreadsheet, the calculated x, y, and z difference (shift) was then applied to other points collected during the survey. Following this operation, a rotation was applied to the surveyed data to correct for small variations in the estimated north direction used to orient the Top Con instrument. The points were then imported into GIS for plotting on a geo-referenced ortho-photograph. The longitudinal profile was then computed and plotted in Excel. During the survey, we used the descriptor "GCS" to indicate portions of the Ross Creek channel that had a grade control structure installed. We did not note the specific design or estimate an age since installation. These points were included in the long profile calculation because they control the local bed elevation and slope, and were also added as an additional series on the long profile chart for illustrative purposes.

A longitudinal profile for San Tomas Creek was not conducted as part of the fieldwork for the HMP and a longitudinal profile for Thompson Creek was constructed using engineer surveys from 1968.

3.4.4 Local channel slopes

Channel slope for each cross-section was measured using an automatic level, stadia rod, and tape. The field crews measured an elevation upstream and downstream from the cross-section over a distance that varied depending on the homogeneity of the site, steepness of the slope, and visibility with the level. Field-measured channel slopes were then compared to regional slopes calculated from existing topographic maps and longitudinal profiles as a way to check the reasonableness of field results.

3.4.5 Channel bank characterization

The stability of channel banks (the ability of channel banks to resist erosion) depends on several factors, including bank material composition and stratigraphy, bank angle, current and former

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[4] The longitudinal survey methods described in Section 3.4.3 for Ross Creek were written by William Lettis & Associates and inserted into this report.

bank vegetation, and subsurface conditions. These parameters were either directly or indirectly (through survey work) described at each cross-section.

At each cross-section, the project team characterized the channel banks by describing bank material; sketching bank stratigraphy; noting the type and density of bank vegetation (including root structure); and describing the types, frequency, and possible causes of bank failures observed at or near the surveyed cross-section.

3.4.6 Channel bed characterization

Channel bed material was characterized primarily to help evaluate channel bed and bank response during different flow events. Quantifying the size distribution of bed material allows the use of equations to calculate entrainment and bedload transport rates, and as a measure of grain roughness in the channel (Lane, 1948; Kondolf, 1997). Two methods were employed to characterize bed material at Thompson, Ross, and San Tomas Creeks—bed core samples and pebble counts. Both methods are quantitative methods used to characterize the distribution of particle sizes for analysis of sediment entrainment and transport.

Bed core samples were taken using a 6-inch (15-cm.) sediment canister (3-lb. coffee can) to collect the bed material sample at a representative location near the cross-section. The sample characterizes the distribution of sediment size in transport at high flows (those able to remove any armoring layer). Samples were analyzed at Cooper Testing Lab in Mountain View, California for particle size distribution (ASTM D-422, w/ #200 wash).

A pebble count, a random sampling of sediment particle-size on the river bed, was conducted at cross-sections that contained sediment too large to be sampled by the bed-core method. Grains were selected using the zig-zag method (modified from Bevenger and King, 1995), measured (along the intermediate axis), and tallied. These data were then used to develop a particle-size distribution. Results are summarized by a calculated median value, which is the size, in millimeters, for which 50 percent of the sample is finer (D-50).

It is important to note that there is a substantial difference in the way that sample statistics are calculated from the two different sampling methods. Bed-core calculations are based on weight, whereas pebble count calculations are based on the proportion of the bed area occupied. Additionally, the bed-core method integrates the sediment stored beneath the bed (also known as 'bed material' or 'bulk bed material'), whereas pebble counts sample only the material at the surface (formally known as 'bed-surface material'), which may be the result of formation of a surface layer or 'armor'. A substantial body of emerging research suggests that the bed-surface population may be different from that just below the surface. Median size descriptors for the two populations (sampled as described above) are used in a number of analytical methods or sediment-transport formulae. While we recognize that these differences exist, we do not quantitatively analyze differences between the surface and subsurface bed material in the particle-size results; this may not be necessary for the purposes of evaluating overall channel stability.

3.4.7 Erosion rankings

Erosion rankings of 'high', 'medium', or 'low' were assigned to each of the cross-sections along Thompson, Ross and San Tomas Creeks by Balance geomorphologists. These rankings were qualitative in nature, which means that no absolute numerical standard was applied. Instead, all factors affecting channel bank and bed stability (channel slope, undercut banks, bank slope, type and density of bank vegetation, in-channel obstructions, knickpoints, etc.) were combined with the observed extent, estimated age, and magnitude of existing erosion to designate an appropriate erosion ranking.

Channel cross sections that received "high-erosion" rankings typically showed signs of active reach-wide undercutting, bank retreat, and/or incision. "Low-erosion" rankings were assigned if the amount of channel erosion (bed and bank) noted at the cross-section was minimal and deemed typical of a natural setting. Therefore, we did not classify all erosion as "medium" or "high," but instead recognized that erosion is a natural process in stream systems. Our goal was to distinguish severe erosion that had occurred at an accelerated rate over an entire reach or sub-reach that would indicate a systematic channel response to a watershed disturbance. Balance field staff also did not select or classify cross-sections manifesting site-specific problems, such as erosion that occurred around an individual stormwater outfall, sackcrete bank protection, or grade control structure.

3.4.8 Photographic record

A set of photographs was taken at each cross-section from at least three different vantage points; across the cross-section, upstream from the cross-section, and downstream from the cross-section. Additional photographs recorded features of interest, such as unique or diagnostic bank stratigraphy, failing grade control structures, and areas where depths of incision from bridge abutments or pipes were measured. This photographic record illustrates problems sites and can also serve as a frame of reference for evaluating the creeks in the future.

3.5 Results

This section describes key results of the geomorphic field work along Thompson, Ross and San Tomas Creeks. The creeks are presented in a consistent format for ease of comparison.[5] The Thompson Creek measurements were completed in early 2003 to aid in the development of the stability assessment model. The Ross Creek and San Tomas Creek work was completed in the late-summer of 2003 to test the model results in different hydrogeomorphic regions of the Basin. For this reason, the Thompson Creek work is the most comprehensive, and work in the Ross and San Tomas subwatersheds was simplified to incorporate a wider geographic range of test cross-sections.

[5] Because of differences in the work scope tailored for each creek, not all information is available for all reaches of all creeks.

3.5.1 Thompson Creek

3.5.1.1 Geomorphic reaches

The project team distinguished ten *geomorphic reaches* based on the longitudinal variation of channel bed and bank conditions along Thompson Creek. The reaches are shown on Figure T3-1 and include seven along the main stem of Thompson Creek and three that describe the major tributaries of Thompson Creek. Within a given reach, broadly-similar influences (whether vegetation, geology, topography, level of upstream development, etc. or some combination therein) affect channel form and processes.

The primary geomorphic features of the seven reaches along the main stem of Thompson Creek are as follows:

- Geomorphic Reach A: Exposed claypan (and/or hardpan) with localized patches of deposition
- Geomorphic Reach B: Incised, with severe bank erosion
- Geomorphic Reach C: Incised, with a sinuous planform, and bank erosion at bends
- Geomorphic Reach D: Channel form presently controlled by dense vine vegetation
- Geomorphic Reach E: Actively incising with bank erosion
- Geomorphic Reach F: Upper end of urbanization, less disturbed with minimal bank erosion
- Geomorphic Reach G: Canyon-like channel with landslides

The three reaches along the tributaries of Thompson Creek are distinguished as follows:

- Geomorphic Reach H: Actively incising with bank erosion and large knickpoints
- Geomorphic Reach I: Stable rock- or clay-cut tributary channels
- Geomorphic Reach J: Engineered channels or storm drains

Table T3-1 summarizes the geomorphic reaches along Thompson Creek, the key characteristics defining each, and how the reaches overlap with the hydrographic segments discussed below.

3.5.1.2 Hydrographic segments

The project team identified six hydrographic segments based on significant flow increases in the downstream direction due to confluences with major tributaries (Figure T3-2). Each hydrographic segment is assumed to have similar stream discharges.

The six hydrographic segments are as follows (listed from downstream to upstream):

- Segment TC1: Thompson Creek between Norwood Creek and Quimby Creek
- Segment TC2: Thompson Creek between Quimby Creek and Fowler Creek
- Segment TC3: Thompson Creek between Fowler Creek and Evergreen Creek
- Segment TC4: Thompson Creek between Evergreen Creek and the confluence of Yerba Buena Creek
- Segment TC5: Thompson Creek upstream of Yerba Buena Creek
- Segment YB1: Yerba Buena Creek

The results of the geomorphic fieldwork are presented by segment. As previously noted, Table T3-1 shows how the geomorphic reaches overlap with the hydrographic segments.

3.5.1.3 Field reconnaissance

The following observations are among those made during the reconnaissance visits:

- Upper Thompson Creek near the headwaters has very erodible banks, a canyon-like form, and is prone to landslides. The channel banks there are composed primarily of shaley material.
- During the November 20, 2002 site visit, most of Thompson Creek was dry. However, portions of the creek were wetted due to urban runoff, particularly near subdivisions upstream from the Yerba Buena confluence. The entire creek showed signs of having sustained flow during the November 8th storm, even though most of it was dry twelve days later; this illustrates the intermittent nature of the stream.
- Dense blackberry vines cover the banks and overhang into the channel of Thompson Creek near the Yerba Buena confluence. This appears to have a significant effect on channel stability, as well as on flow.
- Several old grade control structures, outfall structures, and bank protection projects were observed along the length of Thompson Creek. Many of these structures have failed, and "hanging" structures were deemed a useful tool to measure approximate depths of incision.

3.5.1.4 Channel geometry and channel slope

Channel geometry and slope measurements are summarized in Table T3-2 and described here by segment and cross-section in the upgradient direction. The Yerba Buena Creek tributary is described last. The channel geometry of TC1 is generally variable due to continued changes

induced by the historical and current channel modifications (described in 3.6.1). Moving upgradient from TC1-1 to TC1-5, channel width increases from 16.8 ft to 25.8 ft and depth increases from 4.3 ft to 9.5 ft. Cross-section TC1-6 widens to 52.0 ft with a depth of 11.5 ft and drastically narrows to 11.1 ft and only 3.4 ft deep at TC1-7. Local channel slopes range over an order of magnitude (0.002 to 0.017 ft/ft), varying throughout the reach.

Segment TC2 through TC3 is extremely variable in width and depth. The channel attains its greatest width and depth of 59.4 ft and 13.2 ft, respectively, at TC2-10. In contrast, at TC3-7, the width and depth are 13.6 ft and 3.6 ft, respectively. Channel slope is generally steep, averaging 0.010 ft/ft through Segment TC2, decreasing to an average slope of 0.007 ft/ft through Segment TC3.

Segment TC5 shows a rapid transition in channel geometry, because the upgradient extent of incision is located within this segment. TC5-1 is characterized with a channel width of 46.1 ft and a depth of 11.9 ft. Further upgradient the channel width decreases to less than 20 ft, whereas the channel depth ranges between 1.8 and 7.3 ft. TC5 is located in the uppermost reach of the Thompson Creek watershed and averages a slope of 0.012 ft/ft.

The Yerba Buena Creek tributary increases in width from 4.4 ft, at the upper most cross-section, YB1-0, to 27.2 ft at its confluence with Thompson Creek. Channel depth varies along this reach, but ranges from 0.9 ft to 9.4 ft. The slope of the Yerba Buena tributary is generally steeper than the main stem of Thompson Creek, averaging 0.019 ft/ft.

3.5.1.5 Bank material

Channel banks along Thompson and Yerba Buena Creeks are not homogenous; instead, they consist of multiple layers with different sedimentological characteristics that vary longitudinally through the subwatershed.

In general, the project team identified four distinct types of bank material in the Thompson Creek subwatershed:

- Well-consolidated (or cohesive) laterally-continuous stiff clay/silty clay
- Moderately soft silty/sandy loam
- Colluvium (silty loam, not stiff or dense, and often of limited continuity)
- Alluvial deposits (mixed sands, gravels, some silt and clay)

A stiff clay or silty clay layer was generally found in the lower portion of the bank, extending upward from one to six feet. The same type of bank material was found exposed on the channel bed in Segments TC1 and TC3. This material varies in color from dark brown to brownish-orange (near TC5-2) and reflects that there are at least two units with similar sedimentological composition and erosive resistance, likely deposited at different times historically. The cohesive silty clay/stiff clay layer is very resistant to erosion.

The silty/sandy loam material was found in many cross-sections of the Thompson Creek subwatershed, and varied both in compactness and dominance of silt versus sand. In some cross sections of Segment TC1 the material observed as silty/sandy loam might be artificial fill. Throughout the subwatershed, this layer was usually observed in the upper portions of the banks, where it often holds vertical or near vertical angles.

Colluvial deposits, composed of non-compacted loam, were generally very erodible. This unit is the least continuous of the identified units, as it is found only locally where the creek has cut into slope wash deposited at the base of the stream banks.

Alluvium, former channel bed and bar material, was often observed in the channel banks. These layers, composed of moderately to well-sorted sands and gravels with some silt and clay, often represented the weakest portion of the bank. Widespread undercutting often occurred at the contact between these weaker alluvial deposits and more resistant silty clay or dense sandy loam layer.

3.5.1.6 Bed material

Table T3-3 summarizes the results of the Thompson Creek bed characterization study. D-50, the sediment size that 50% of a sample is finer than, ranged from 3.2 to 10.2 millimeters, with the smallest value from the upper portion of Segment TC2 and the largest value from upper Yerba Buena Creek.

D-50 values are quite varied in the upper portion of Segment TC5 (samples from cross-sections TC5-5 through TC5-7), reflecting the diverse characteristics of this section of TC5. The low D-50 value at cross-section TC5-7 is possibly influenced by debris flow material from the upper subwatershed.

The D-50 value at cross-section YB1-7 is finer than that at YB1-1. However, the bedload sample collected near YB1-7 during the December 14, 2002 storm (YBSF)[6] is more similar to the bed material at YB1-1. This may be due to recent deposition of sand at the lower cross-section since the December 2002 storms, or an historic shift in the material being deposited at the cross-section.

The small (micrometer) D-50 numbers in Table T3-3 reflect the high clay and silt content of the cohesive bed material that was sampled at select cross-sections in Segment TC1, TC3, and Yerba Buena. At these cross-sections, the claypan was exposed and a grab sample was collected for hydrometer analysis. Our intention was to evaluate whether the cohesive bed material exposed in Segment TC1 was similar to TC3, and how both of these compared to the cohesive bed material found along Yerba Buena Creek. Results indicate that the cohesive bed material collected at TC1-3 and TC3-6 are very similar in clay, silt, and sand content, both being primarily composed of silt (approximately 45 percent) with a significant clay component (greater than 30 percent). The Yerba Buena sample (taken at YB1-6) contains more sand and less clay than the samples from Segments TC1 and TC3, resulting in a slightly higher D-50.

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[6] See methods section for discussion of bedload sampling.

3.5.1.7 Vegetation

The vegetation within the Thompson Creek subwatershed consists mainly of grasses, trees (most frequently bays, oaks, and some eucalyptus and maples), blackberries, and poison oak. Some sections of the creek also have a thin layer of moss that is partially stabilizing bank slopes.

Trees and blackberries probably play the most important role in slope stabilization. In Segment TC4 and the lowest portion of Segment TC5 the blackberry vines are so thick that the plants—not the actual banks—define the channel dimensions. In the upper portion of Segment TC1 and throughout Segment TC2, severe bank erosion often stops where there are large trees with a deep root structure that stabilizes the banks. In general, vegetation density below the top of the banks decreases downstream. The lower portion of Segment TC5 and the upper portion of Segment TC4 have the densest vegetation, with some portions having in-channel vegetation. The lower portion of Segment TC1 has very little vegetation, with only grasses and small trees on the top of the banks.

Riparian corridor width also decreases downstream. This affects stream bank stability, because one row of trees provides less stability than a gallery or a more developed riparian corridor. The vegetated banktop riparian corridor width is narrowest in Segment TC2, where roads, fences, and parking lots are present at the top of the stream banks.

3.5.1.8 Erosion Rankings

Erosion rankings of high, medium, or low were assigned to each of the cross-sections along Thompson Creek. A rank of "stable" was assigned to two upper sections on Thompson Creek and one on Yerba Buena Creek.

Cross-sections that received an erosion ranking of "high"' were typically severely incised or actively incising with actively eroding banks, either through processes of undercutting, shearing and/or slumping. Channel banks were generally vertical and bare, with vegetation stabilizing only the very tops of the banks. A few cross-sections designated as "high" did not display tall, vertical banks (such as TC5-5) and with brief visual inspection appear in relatively good condition; however, recently exposed roots on the channel bed and banks indicate that incision is just starting to occur in these cross-sections on a reach wide scale. Without change in the existing magnitude and frequencies of flow, it is likely that these creek sections will experience further incision and bank erosion comparable to cross-sections located in older incised portions of the creek that also received "high" designations.

In total, nine of the thirty-seven cross-sections were assigned a "high" erosion ranking, with two additional cross-sections bordering between "medium" and "high." Cross-sections with "high" erosion rankings are located throughout the Thompson Creek subwatershed, but particularly concentrated in the lower portion of Segment TC5, just downstream from some of the most recently-constructed housing.

Cross-sections were the erosion ranking was designated "low" typically exhibited banks that were either not eroded or eroded but now in a stable configuration. Also, 'low' cross-sections generally had gentler slopes and denser vegetation, and sometimes were located behind grade

control structures where the bed had aggraded. Cross-sections ranked as "medium" had characteristics in between those described for "high" and "'low." Thirteen out of thirty-seven cross-sections were designated as "medium" and thirteen designated as either "low" or "low" to "medium."

The two uppermost cross sections on Thompson Creek (TC5-6 and TC5-7) were assigned a "stable" condition as well as YB-0 on Yerba Buena Creek. Stable conditions are defined in Chapter 5.

3.5.1.9 Segment characteristics

Segment TC1 - Thompson Creek between Norwood Creek and Quimby Creek

Segment TC1 of Thompson Creek (between Quimby and Norwood Creeks) is the most downstream segment of Thompson Creek included in the field survey. This segment has engineered flood control levees on both sides of the channel, with significant bank and instream structures. Based on aerial photographs, it appears that the lower portion of Segment TC1, below cross-section S1-6, was straightened sometime prior to 1968. The bed in Segment TC1 has incised in the past, leaving raw, vertical banks in many places. The amount of incision in this segment is *at least* three feet, based on measurements of a former subsurface pipe that is now exposed in the bank; it is unknown when major incision began. The incision has been effectively halted by a claypan layer that now forms the channel bed in many portions of the segment. The clay layer resists erosion; it appears to incise slowly by the process of shear and abrasion rather than by knickpoint propagation, as is seen in Yerba Buena Creek and the upper reaches of Thompson Creek.

Currently, grade control structures are modifying the pattern of bed erosion and deposition in Segment TCI, as follows:

- A weir installed at the Quimby Road crossing has generated localized aggradation that extends about 150 feet upstream behind the weir.

- Upstream from the Quimby Road weir, a large sackrete grade control structure has been installed in the channel (see Figure T3-5).

- A series of four boulder cross-vane grade control structures were recently installed in the middle of Segment TC1 as part of a channel stabilization/restoration project. Sediment has already started to accumulate behind these structures.

- In the middle to upper portion of the segment, an old partially destroyed weir is still acting as a small grade control structure and creating a zone of localized aggradation.

Although Segment TC1 has localized zones of aggradation, much of the segment is still experiencing erosion. Most of the active erosion is targeted at the banks, with undercut and slumped banks common features throughout the reach.

Gravel deposits or hard clays make up the bed material throughout Segment TC1. The gravel often appears as a thin veneer in the lower to mid portions of the segment, where the 'claypan' is either directly exposed or is covered with only three to four inches of gravel. The depth of mobile material in this segment rarely exceeds 12 inches, and when it does, it is usually behind grade control structures such as the Quimby Road weir. Most of the gravel is fairly transient, moving downstream with each significant flow event. Little to no pool-riffle morphology is found, although the bed is slightly more complex at the upper end of the segment.

The banks are composed primarily of hard clay near the channel bed and a thick silty loam layer (possibly artificial fill) extending to the top of bank. At some sites, sand and silty gravel deposits are interspersed between the hard clay and fill material. The weakest layers are these alluvial deposits composed of sand and gravel. Undercutting occurs at the interfaces between the weak sandy gravel and the more resistant clay and fill layers (see Figure T3-6).

Grasses and small woody vegetation make up most of the bank vegetation along the lower portion of Segment TC1 near Quimby Road. Larger trees become more common near the confluence with Quimby Creek. The deep roots of these larger trees aid in stabilizing some of the eroded banks in the upstream portions of Segment TC1.

Segment TC2 - Thompson Creek between Quimby Creek and Fowler Creek

Segment TC2 (just upstream of segment TC1, between Quimby and Fowler Creeks) can be divided into two sub-segments based on geomorphic characteristics, with the break at the Aborn Road crossing. The lower portion of TC2 is very similar to the upper portion of TC1 (above TC1-5), with steep banks about ten feet high and many areas with vertical, actively eroding faces. Undercutting and slumping of banks is common in this sub-segment; mature trees are growing below the top of the banks, partially stabilizing the banks in some areas.

The portion of Segment TC2 above Aborn Road has a slightly higher sinuosity than that downstream. In general, vegetation is denser than in the lower portion of Segment TC2 and, partially because of that, the channel banks are more stable (see Figure T3-7). Several areas with severely eroding banks are present within this section of Segment TC2, mostly at outside bends in the creek, though they are not as extensive or as common as those downstream. Development (roads, parking lots, fences) in this area is often very close to the top of the stream bank, which makes bank stability of particular concern.

Throughout Segment TC2, the lower portion of the channel banks are typically composed of clayey sand and gravel to moderately well-sorted sand and gravel. Above this is a layer of dense, poorly sorted material about five feet thick, probably fill material underlying the surrounding developed areas. Because the fill has been compacted, it is slightly more resistant to erosion than the lower layers; as a result, there is undercutting (up to a foot) below the fill layer.

The bed in the upper portion of Segment TC2 is more complex than segment TC1 and has distinct pools (deeper areas) and bar forms, likely due to its meandering planform. Depth of mobile sediment is generally greater than one foot, but is highly variable because of the more complex bed morphology.

A grade control structure is present at the downstream end of the Aborn Road crossing; the structure arrested any downcutting that may have been occurring and probably caused local aggradation upstream from the structure. There is also a concrete slab (probably an old bridge crossing) upstream from cross-section TC2-6 that provides a grade control for the upper portion of this segment.

Segment TC3 - Thompson Creek between Fowler Creek and Evergreen Creek

Segment TC3 is very similar morphologically to the upper portion of Segment TC2 and as a segment is fairly homogenous in terms of bed and bank characteristics, as well as channel geometry. Like the upper portion of Segment TC2, the channel planform is quite sinuous, exhibiting in places very tight meander bends (see Figure T3-8). Severe erosion is occurring at the outside of meander bends where the creek is close to the road, including in some sections where the creek is constrained by a retaining wall.

The bed consists of a mixture of sand, gravel, and some cobbles. The median grain size is mostly fine gravel. The bank material throughout the segment consists of a silty pebbly loam or silty clay loam with some embedded gravels.

Segment TC3 is characterized by very dense vegetation, with more shrubs and vines than trees. There is a significant amount of debris in the channel, resulting in shifts in flow path direction, scour holes, and bank erosion. The types of debris found in the channel include telephone poles, riprap, old bridge structures, remnants of previous retaining walls, and trees.

Segment TC4 - Thompson Creek between Evergreen Creek and Yerba Buena Creek

Segment TC4 was not included in the field survey because no problem sites identified by SCVWD were located there and dense vegetation made access to the segment very difficult. However, based on the project team's familiarity with the upstream and downstream segments, and some observations during early reconnaissance walks, it appears that Segment TC4 shares similar qualities with Segment TC3, at least in the lower portion. Near the Yerba Buena confluence, the banks are covered with blackberry vines, as occurs in the downstream-most sections of Segment TC5.

Segment TC5 - Thompson Creek upstream of Yerba Buena Creek

Segment TC5 (located upstream of the Yerba Buena Creek confluence) includes the headwaters of Thompson Creek, however the most upstream cross-section surveyed was located at Early Morning Drive (TC5-7), a little over two miles upstream of the confluence with Yerba Buena Creek. Upstream from this cross-section the watershed remains relatively undeveloped, with only scattered homes and roads.

From cross-section TC5-7 to at least cross-section TC5-6 the creek has been relatively unaffected by development except where it is crossed by roads. The banks are well vegetated (mainly with grasses and trees), there has been little or no incision, and the banks are stable throughout most of the section. Some slumping of the banks has occurred, primarily where the creek is confined against the valley wall by San Felipe Road.

The portion of Segment TC5 just upstream of the bridge crossing near The Villages Parkway subdivision, at cross-section TC5-5, is characterized by thick vegetation, and a straight channel with a relatively steep local slope. Bed degradation is beginning at this cross-section, as there are many exposed roots and root mats at the base of the channel banks and on the stream bed (see Figure T3-9).

Downstream of cross-section TC5-5 the stream generally shows an increase in incision and bank degradation. Between cross-sections TC5-2 and 5-3 is an old bridge crossing dated 1947. The creek has down-cut about 3.2 feet below what looks to be the original base of the abutment (see Table T3-4).

Several portions of lower Segment TC5 are now underlain by a tan, compact clayey silt, similar to that seen in Segments TC1 and TC2. This has certainly slowed incision, and provides some stability for the lower portions of the banks (see Figure T3-10).

The section of creek just upstream of Yerba Buena is characterized by dense vegetation, mostly blackberries, poison oak, and shrubby trees. In one section, the blackberries were so dense that they seemed to be the primary feature channeling high flows, rather than the banks. Because of this dense vegetation, no cross-sections were surveyed in this portion of the segment.

Segment YB1 - Yerba Buena Creek

Segment YB1 is Yerba Buena Creek, a tributary to Thompson Creek. The two streams join at the upper end of Segment TC4. Yerba Buena Creek is actively incising throughout most of its length, except for the uppermost sections of the creek, above the golf course and the upstream-most urban-area outfalls, and possibly near the confluence with Thompson Creek where it is locally aggrading.

Segment YB1 is one of the most complex and diverse segments in the Thompson Creek subwatershed. This segment has several large knickpoints that divide it into several sub-segments with different geomorphic characteristics, especially with respect to channel geometry. The characteristics of this segment are described from upstream to downstream.

The portion of Yerba Buena Creek upstream of Villa Vista Road is in a fairly stable configuration, with only minor bank erosion evident at localized sites. This section of Yerba Buena Creek is comparable to the upper portions of Thompson Creek, described for Segment TC5. The bed consists of angular to subangular coarse material (mainly coarse gravel with some cobbles), and these materials seem to be readily entrainable during high flows. The upper portion of Yerba Buena Creek is not as sinuous as the downstream section. Most of the banks are composed of the pebbly silty loam complex, with the exception of one location where fine-grained sedimentary rock is exposed. The bed slope is gentle and the banks appear stable, although the bed is incising progressively downstream from the first outfall.

The first large knickpoint (with an approximate drop of seven feet) in a series of knickpoints along Yerba Buena Creek occurs just downstream from this upper section, approximately 50 feet downstream from the new SCVWD sediment sampling and gage site near the upstream end of Villa Vista Road (YBVV). This knickpoint has temporarily stopped migrating upstream due to a sackrete sill built across the channel at the location of an outfall structure. The knickpoint may

be associated with additional stormwater runoff from this outfall structure, which is located just at the head of the knickpoint.[7]

Downstream of Villa Vista Road, another outfall structure with sackrete is being threatened by significant incision. Downstream from this incision, the channel recovers back to the shape similar to the more stable upstream reach, although wider. At some locations, it is also deeper. In general, the banks average two to three feet.

Continuing downstream, the channel becomes tightly confined, with deep, vertical banks. There are two knickpoints in the section between Silver Estates Road and Creek Estates Road. The first knickpoint is located between the two roads. It has a drop of approximately five to six feet. Upstream from this knickpoint, the channel is about six feet deep and six feet wide. Downstream from the knickpoint, the channel is approximately 12 to 15 feet deep. Further downstream, immediately after the pedestrian bridge west of Creek Estates Road, there is another knickpoint with a drop of approximately eight feet. At this portion of Yerba Buena Creek, near cross-section YB1-5, the channel is deeply incised (approximately 20 to 25 feet) and very narrow (see Figure T3-11), and hard clays are exposed on the channel bed. This narrow and deep section is relatively short (about 700 feet) and ends downstream of Sunny Creek Road.

Downstream from Sunny Creek Road, the channel reverts to a geometry with five to six feet of depth and approximately eight feet of width, and meanders tightly. Further downstream and immediately upstream of the confluence, the channel is wider and the banks are shallower.

The bed material varies between fine gravel and coarse gravel depending on the sand and cobble content. There is significant local supply throughout the segment, with active transport of sediment from the upper watershed to downstream portions of the creek.

The large knickpoints in Yerba Buena Creek will likely continue to propagate upstream and therefore Yerba Buena Creek will continue to supply high amounts of sediment to Thompson Creek.

5.5.1.10 Discussion

This section of the report discusses what the field observations may mean, primarily with respect to:

- Speculating on how these findings might be extenable to other subbasins in the Thompson Creek catchment and other watersheds in the Santa Clara Basin.
- Developing useful local criteria for managing hydromodification
- Working toward design criteria for evaluating likely future channel configurations

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[7] During each of the significant rainfall events during water years 2003 and 2004, we repeatedly found discharges of more than three cubic feet per second (cfs) emanating from this outfall during rainfall events, while the channel upstream remained dry. Even when it was not raining up to 0.25 cfs of was flowing from the outfall structure.

Range of values

The project team compared the values of the data collected for Thompson Creek to data recorded from other streams. Some parameters, such as slope and width-to-depth ratio, appear broadly typical of other stream systems in the region that are affected by hydromodification.

Width-to-depth ratios recorded in Thompson Creek are typically in the range of three to five, compared to ratios of four to six reported for channels throughout clayey soils in southern Contra Costa and central Alameda Counties. The difference is not large and may be due to recent incision along Thompson Creek and the fact that the banks there have not yet retreated to a semi-stable position. The field measurements and reconnaissance indicate that most channel banks in the Thompson Creek subwatershed can hold steep, nearly vertical angles. This is especially true in Yerba Buena Creek. It is possible that some sections of the creek may not retreat into a semi-stable position in the near future and will instead preferentially incise instead of widen (such as the canyon-like banks of Yerba Buena Creek where the banks are over 20 feet deep – see Figure T3-11). This is most likely to occur where the banks are composed of very cohesive clay and silt material.

Data may not be directly applicable to streams with non-cohesive banks, where width-to-depth ratios of 8 to 20 are more typical. Coyote Creek and reaches of Uvas and Llagas Creeks are examples of such the latter type of channel.

Stable versus unstable cross-sections

The Thompson Creek subwatershed is fairly diverse in terms of channel form, geology, hydrology, vegetation, and the classes of erosion problems manifested, which is reflected in the classification of ten distinct geomorphic reaches along Thompson Creek and its major tributaries. Even with such complexity, much of Thompson Creek (except the upper portions of Thompson and Yerba Buena Creeks) is considered unstable or excessively eroding, where the hydrology has changed and the existing channel form is no longer stable.

The terms "stable" and "unstable" are often used as a shorthand when conducting geomorphic assessments, with stable indicating that all forces (erosive and resisting) are in balance, or equilibrium. Generally, the designation of a cross-section, site, reach or watershed as stable or unstable is qualitative and based on visual inspection (see Chapter 5 for a discussion of stability).[8]

The field crew identified three stable cross-sections during the field campaign: YB1-0, TC5-6 and TC5-7. These cross-sections can be cautiously extended to state that Geomorphic Reaches F, G and I are 'stable' reaches. Geomorphic Reaches F and G are located above most existing development (Figure 3-1). Downstream changes in channel form as a result of

[8] References throughout this report to sites or reaches as 'stable' or 'unstable' are solely for the fluvial geomorphic planning-level analysis underlying this report. They are expressly not intended to substitute for the detailed site geotechnical or slope-geomorphic investigations required prior to site evaluation or design of facilities and structures.

hydromodification have not yet influenced these upstream reaches. Geomorphic Reach I is stable because the channel bed in this reach is cut into bedrock or hard, dense clay. It is also possible that Reach D is stable due to the dense vegetation cover that is protecting the channels banks throughout the reach. However, no cross-sections are located in this reach and visual inspections of channel bed stability were not conducted.

The remaining thirty-four cross-sections, including all cross-sections in Segments TC1, TC2 and TC3 were identified as unstable. The lower portions of Thompson Creek have experienced incision of *at least* three feet and as a result channel banks are steep, bare, and failing in many places. Although some individual cross-sections appear in a relatively stable configuration, there is high potential for instability to develop. Active incision is still occurring along the lower to mid portion of Segment TC5 and in Yerba Buena Creek, causing the cross-sections located in these reaches to be defined as 'unstable.'

Role of vegetation

Vegetation appears to be a significant stabilizing element for many of the channel banks along Thompson and Yerba Buena Creeks. In the Thompson Creek subwatershed, the principal means by which vegetation stabilizes the channels appears to be:

- Banktop vegetation. Such vegetation holds the looser soils in place, and the roots strengthen the banks or (when exposed to high flows) reduce the velocities impinging on the bank. The banktop vegetation often includes large coast live oaks or bays, but sometimes includes deciduous trees or brush.

- Roots as grade control. Root systems or major roots of larger trees frequently serve to inhibit incision, even sometimes serving as a local base level. With a very few exceptions, roots inhibit incision only in channels up to a certain size. Stabilizing roots are found on upper Thompson Creek beginning perhaps one mile upstream of the Yerba Buena confluence, and also on Yerba Buena Creek as far downstream as the lower end of Evergreen College.

- Smaller woody vegetation. Vines and brush are the predominant stability-lending element in some of the channels, at least up to certain thresholds. Thompson Creek upstream of the mouth of Yerba Buena Creek provides a good example of stabilizing brush covering the banks.

- Small woody debris jams. While small woody debris jams temporarily stabilize the channel, their effects are considered transient and are not as long-lasting as those imparted by large-conifer logjams.

The role of vegetation in stabilizing the banks and bed is considered in the stability assessment through the selection of the hydraulic roughness coefficient. Higher values of roughness slow velocity and reduce shear stress on the boundary.

3.5.2 Ross Creek

3.5.2.1 Geomorphic reaches

The project team distinguished six *geomorphic reaches* based on the longitudinal variation of bed and bank conditions along Ross Creek; five along the main stem of Ross Creek, with the tributary of East Ross Creek identified as a distinct reach. The reaches are shown on Figure R3-1 and further described in Table R3-1.

The primary geomorphic features of the six reaches along Ross Creek are as follows:

Geomorphic Reach A:	Earthen trapezoid channel with meandering low flow channel and in-channel "bench"
Geomorphic Reach B:	Earthen trapezoid channel with steep channel banks and severe bank erosion (widening)
Geomorphic Reach C:	Earthen trapezoid channel punctuated with grade control structures
Geomorphic Reach D:	Non-engineered, 'natural' channel with numerous bank protection projects, low erosion
Geomorphic Reach E:	Non-engineered, 'natural' channel with active bank widening
Geomorphic Reach F:	East Ross Creek; a headwaters tributary

3.5.2.2 Hydrographic segments

Hydrographic segments were not defined for Ross Creek due to the high number of stormwater outfalls that discharge to the channel over small intervals of channel length. Clear increments in flow volume were not as distinguishable in the channel profile as with Thompson Creek, which had several tributaries and large outfalls that marked clear transitions between different hydrographic segments.

3.5.2.3 Field Reconnaissance

The following observations are among those made during reconnaissance visits to Ross Creek in July 2003:

- Approximately two feet of consistent incision is observable in the lower, channelized[9] reaches of Ross Creek. This incision occurred sometime after channelization during the mid-

[9] The terms 'channelized' and 'channelization' are used in this report to mean 'engineered' (see discussion in Knighton, 1998). In the specific case of Ross Creek, the 'channelized' reaches are those that were altered in the late 1950's for flood control purposes. The 'channelized' portion of Ross Creek extends from the confluence with the Guadalupe River upstream to Camino Del Cerro. Similarly, 'unchannelized' is used to describe the unmodified, 'natural' reaches of creek upstream from Camino Del Cerro and including most of East Ross Creek.

to late-1950's and has in many instances been halted due to local grade control or, where active erosion is minimal, the establishment of a new quasi-equilibrium form.

- The former channel bed of the engineered channel is now a distinct linear "bench" along much of the channelized portion of Ross Creek.

- The lower, engineered reaches of Ross Creek do not support a riparian corridor, as the channel is often confined between two gravel maintenance roads at the top of the banks. Some vegetation, mostly grasses, forbs and some shrubs, cover the channel banks, but trees are rare. All bank vegetation along the lower extent of the main stem of Ross Creek (where maintenance roads exist) is regularly mowed to maintain flood capacity.

- At the time of our reconnaissance in July 2003, Ross Creek was flowing throughout much of its length. The stream was dry below Cherry Avenue, but flowing throughout the upper reaches. East Ross Creek was dry in the upper portion of its watershed (at ERC-3), but flowing through much of the lower portion of the watershed (at and below ERC-2).

3.5.2.4 Channel geometry and channel slope

Several channel geometry parameters were calculated from the cross-section survey data for Ross Creek, including top-of-bank (TOB) widths, average depths from TOB, width-to-depth ratios, and in relevant cross-sections, widths and depths of natural or artificial in-channel "benches." Top-of-bank was used as a reference for width and depth measurements because bankfull dimensions, which are geomorphically more meaningful in self-formed alluvial channels than TOB dimensions, were not relevant in the highly modified Ross Creek system. The top of bank for each cross-section was identified in the field where possible, and interpreted from plotted cross-sections when field data did not explicitly designate a TOB. The top of bank can roughly be defined as the first significant break in slope the field crew encountered when conducting the cross-section surveys. In the lower reaches, this generally corresponded to the edge of the gravel maintenance roads. Channel geometry and local channel slope measurements are summarized in Table R3-2.

Top-of-bank widths at cross-sections along the main stem of Ross Creek range from approximately 24 to 41 feet. The channelized cross-sections have the widest channels from TOB, averaging around 38 feet. This reflects the engineered channel form that was initially constructed in the mid- to late-1950's and perhaps subsequent channel maintenance and modifications[10]. East Ross Creek has smaller TOB widths than the main stem of Ross Creek, with the upstream-most cross-section, ERC-3, having a TOB width of 4.3 feet.

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[10] Although available, engineering drawings from the 1950's showing the original designs planned for the channelized portions of Ross Creek offer minimal assistance for evaluating channel change over time. It appears that the design drawings either substantially differed from what was actually constructed or that subsequent large-scale maintenance and/or repair activities altered the channel from the original design so that comparisons to current conditions are not relevant. For instance, the design drawings suggest that TOB widths were to range from 18 to 22 feet, which is over ten feet less than what exists today. No field observations made by Balance staff suggest that

Channel depths from TOB range from 6.0 to 10.4 feet in the main stem of Ross Creek, and from 0.7 to 6.5 feet in East Ross Creek. The channelized reaches of Ross Creek are the deepest, with depths of around nine feet. Width-to-depth ratio (top-of-bank width divided by mean depth) is a channel geometry parameter that can be used to compare different reaches within the same stream system to evaluate incision and widening. Typically, a high width-to-depth ratio represents a wide and shallow channel and a low width-to-depth ratio represents a narrow and deep channel. What constitutes a "high" value compared to a "low" value is dependent on the characteristics of the watershed and will vary by soil type and region. Width-to-depth ratios for the channelized cross-sections in Ross Creek all calculate to about 4.0, which is not surprising due to the relative uniformity of engineered, trapezoidal channels. Width-to-depth ratios for the unchannelized cross-sections RC-7 and RC-8 are 4.3 and 3.8 respectively and width-to-depth ratios for lower East Ross Creek are 3.0 for cross-section ERC-1 and 4.1 for ERC-2. Only cross-section ERC-3 has a width-to-depth ratio that deviates from 4.0 by more than 1.0, at 6.4. Width-to-depth ratios are fairly uniform for most of Ross Creek, especially when compared to the range of width-to-depth ratios for nearby watersheds, such as Thompson and San Tomas Creeks.

A prominent, linear "bench" was noted at several cross-sections in the channelized reaches of Ross Creek, specifically at cross-sections RC-1, RC-2, RC-4, RC-5, and RC-6. The "bench" is nearly unbroken and appears to be a remnant of the original engineered channel. The depth from the top of the "bench" to the channel bed ranges from approximately 2 to 3 feet in cross-sections RC-1, RC-2, RC-4, and RC-6. This corresponds to the amount of incision Balance staff measured at various culverts and outfall structures located in these reaches suggesting that the observed "bench" is actually the former channel bed of the original engineered channel. At the one cross-section, RC-5, where the depth from the "bench" to the channel bed exceeds 4 feet, design drawings from the 1950's show a mid-bank bench incorporated into the design.

Local channel slopes range from 0.0039 to 0.0067 (ft/ft) in the channelized reaches of Ross Creek. Slopes in the unchannelized reaches of Ross Creek are 0.0145 and 0.0078 for cross-sections RC-7 and RC-8 respectively. Cross-sections in the East Ross Creek tributary have local channel slopes that increase upgradient from 0.0098 to 0.0191. Since all of Ross Creek was flowing during our surveys, water surface elevations were used to measure local slope unlike Thompson Creek where all local slopes were calculated from bed topography. Chapter 5 describes how these local slopes were applied in the stability assessment model.

3.5.2.5 Bank material

Channel banks along Ross Creek vary longitudinally through the subwatershed, but the types of bank material present in Ross and East Ross Creeks can be generalized into several types:

- Silty sand and gravel with some cobbles (alluvial deposits)

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channel banks have retreated over ten feet to their current position. Also, current channel depths from TOB are up to two feet shallower than what is shown in the design drawings, yet we found evidence of channel bed incision (not aggradation) over much of the channelized length of Ross Creek.

- Poorly consolidated fine sand, silt, and clay (overbank deposits)
- Poorly sorted silt, sand, clay, and gravel (construction fill)
- Poorly sorted clay, silt, sand, and gravel (debris flow deposits)

Throughout the engineered section of Ross Creek, the upper portion of the banks is predominately composed of engineering fill material. This fill typically overlies layers of alluvial deposits, with varying amounts of gravel and cobbles. The non-engineered portions of Ross Creek have banks composed of similar alluvial deposits, without the overlying layer of fill material.

The bank material in the upper portion of East Ross Creek is almost exclusively old debris flow deposits, often a matrix-supported mixture of clay, sand, silt, and gravel, with some angular cobbles. At ERC-1, however, there are several feet of engineering fill material on top of debris flow/alluvial deposits.

3.5.2.6 Bed material

Table R3-3 summarizes the results of the Ross Creek bed characterization study. D-50 values, the sediment diameter, in millimeters, for which 50 percent of the sample is finer, ranged from 1.7 to 23 millimeters. The lowest value was from a sample taken at ERC-3, the cross-section highest up in the watershed, and consisted predominately of sand. The other four samples had median (D-50) sizes similar to one another (between 15 and 20 millimeters), and were composed mainly of gravel-sized particles. Differences in these values may in part be due to differences in sampling methods.

Depth of mobile bed material in Ross and East Ross Creeks (where measurable) was generally less than two feet, ranging from six to eighteen inches. In the lower reaches, mobile material was underlain by moderately well-consolidated fine sand and silt. In the upper reaches the underlying material was a cohesive mixture of clay, sand, and silt.

3.5.2.7 Vegetation

The vegetation within the Ross Creek subwatershed is sparse in the lower reaches of the main stem of Ross Creek from the confluence with the Guadalupe River upstream to Camino Del Cerro. These channel reaches have been modified and constructed into a trapezoidal shape. Gravel maintenance roads border both sides of the channel on top of the channel bank. Bank vegetation is maintained through mowing and trimming activities on a regular basis in these lower reaches of Ross Creek. Little to no woody vegetation is present, with grasses and some shrubs most prevalent. Vegetation has become established on in-channel benches and/or bars where erosion is minimal.

A riparian corridor is first evident at the transition from earthen trapezoid channel to the "natural" channel upstream from Camino Del Cerro. Bank vegetation density and riparian corridor width generally increase in the upstream direction toward the headwaters and where residential development decreases in density.

3.5.2.8 Erosion rankings

In total, 6 of the 11 cross-sections were assigned a "medium" or "high" erosion ranking, with one additional cross-section bordering between "medium" and "high." Cross-sections with "medium" or "high" erosion rankings are located throughout the Ross Creek watershed, but are especially clustered in Geomorphic Reaches B and C, where the earthen trapezoid channel is eroding and has yet to reach a quasi-equilibrium state as observed in Geomorphic Reach A.

Cross-sections where the erosion ranking was designated "low" typically had banks that were either not eroded or eroded but now in a stable configuration. Two out of 11 cross-sections were designated as "low to medium" erosion and two designated as "low" erosion. The two cross-sections classified as "low to medium" erosion are RC-1 and RC-2, which are the downstream-most cross-sections evaluated. The bed at these cross-sections, and all of Geomorphic Reach A, has experienced incision since channelization in the late 1950's, but now seems to be in or approaching a quasi-equilibrium state. A low flow meandering channel has developed within the channel and bars have formed that support vegetation in the summer. Active incision is not apparent and bank erosion, although still occurring, is not of the magnitude observed upstream.

Only cross-section ER-1 was deemed "stable" in the Ross Creek subwatershed. Stable conditions are defined in Chapter 5.

3.5.2.9 Geomorphic reach characteristics

Geomorphic Reach A

Geomorphic Reach A extends from the confluence of Ross Creek with the Guadalupe River to Meridian Avenue and is characterized as having an earthen, trapezoid channel with a meandering low-flow channel located within the engineered channel. The channel in Geomorphic Reach A was constructed into its current engineered form in the late-1950s prior to large-scale urbanization of the watershed. The current channel path does not follow the historical, 'natural'[11] path of the channel.

Geomorphic features observed in the reach include pools and riffles that occur at fairly regular intervals. This reach is considered meta-stable (at or approaching a quasi-equilibrium form), with recent bed incision and bank widening evident, but not currently active on a reach-wide scale. It is likely that the 1.5 to 2.0 feet of incision observed throughout the reach occurred as a response to a) channelization in the late 1950's and/or b) the urbanization of the watershed soon thereafter. A mid-bank linear bench is found in Geomorphic Reach A at a relatively consistent height above the existing channel bed (Figure R3-2). This linear bench is likely the former channel bed at the time of initial channelization. Subsequent incision lowered the bed approximately 2 feet below

[11] Although we use the term 'natural' when referring to Ross Creek prior to channelization, the channel at that time (1950's) had already been modified in sections, especially near bridges and due to some farming practices. Therefore, we do not mean to indicate that Ross Creek was completely undisturbed at that time, rather it still retained a meandering planform and had not yet been engineered at a reach-wide scale.

the linear bench, which is also consistent with measurements of "hanging" structures found within the reach.

As with most of the channelized reaches of Ross Creek, gravel maintenance roads border both sides of Geomorphic Reach A and vegetation is sparse and mainly limited to grasses and other non-woody vegetation.

Bank material in Geomorphic Reach A consists of a bottom layer of sandy silt with some gravels and a top layer of fill material. The bottom layer varies in thickness throughout the reach, and in some instances has distinct bedding, suggestive of a former stream channel bed. The engineered fill layer is composed of loose silt and gravels and is prone to gullying. Median size (D-50) of bed material in Geomorphic Reach A is approximately 15 mm.

Geomorphic Reach B

Upstream from Meridian Avenue to Camden Road is defined as Geomorphic Reach B. Several large stormwater outfalls enter the channel in this reach, which is characterized by actively eroding channel banks (Figure R3-3). Like Geomorphic Reach A, this reach has an earthen, trapezoid channel constructed in the mid- to late-1950's. However, unlike Geomorphic Reach A, the engineered channel occasionally follows the former 'natural' channel path (Santa Clara County Flood Control District, 1955). Where the former channel intersects the engineered channel is often an area of instability.

The active channel bed in Geomorphic Reach B is considerably wider than the other channelized reaches along Ross Creek. And the prominent "bench" observed in Geomorphic Reaches A and C is almost completely absent.

Bank stratigraphy consists of a lower layer of sandy silt that is relatively more resistant to erosion than the upper layers. A middle layer, approximately 2 to 3 feet thick, is composed of alluvial material that is mostly sands and gravels. This middle layer is often observed where the former 'natural' channel bed intersects the engineered channel and is the weakest layer, being the most susceptible to erosive flows. The upper-most layer is engineered fill typical of the channelized reaches. The mechanics of bank erosion in Geomorphic Reach B consists of active shearing and undercutting of the middle alluvial layer and resultant slumping of the upper fill layer.

Channel bank vegetation is sparse and regularly maintained by mowing and trimming throughout the reach. Gravel maintenance roads border both sides of the channel. Median size (D-50) of bed material in Geomorphic Reach B is approximately 15 mm.

Geomorphic Reach C

Geomorphic Reach C is located between Camden Road and Camino del Cerro, and is also an earthen, trapezoid channel constructed in the late 1950's. In this reach, the engineered channel intersects the former 'natural' channel at several locations because the engineered channel was constructed in straight segments roughly centered on the meandering 'natural' channel path (Santa Clara County Flood Control and Water Conservation District, 1957).

Geomorphic Reach C is characterized by several grade control structures that have experienced failure (Figure R3-4). Reach-wide bank widening is prevalent although less severe than the bank erosion observed in Geomorphic Reach B. Approximately 1.5 to 2.0 feet of recent or historical incision is noted, but the incision does not appear active and is controlled by the grade control structures.

The linear mid-bank "bench" present in Geomorphic Reach A is again found in Geomorphic Reach C. Bank material and stratigraphy vary throughout the reach based on whether the area surveyed intersects the former channel, was constructed through the floodplain of the former channel, etc. Regardless of the bottom-most layers, the top bank layer is consistently fill material as with the other channelized reaches. Median size (D-50) of bed material in Geomorphic Reach C is approximately 20 mm, slightly larger than the downstream reaches of Ross Creek. Channel banks have minimal grassy vegetation and provide little in the way of bank stabilization.

Geomorphic Reach D

Geomorphic Reach D is a relatively small reach extending upstream from Camino del Cerro to upstream of Linda Court. Camino del Cerro marks the transition from an earthen, trapezoid channel to a less modified channel form. The channel in Geomorphic Reach D is confined between residential neighborhoods on both sides, however, still has a narrow riparian corridor with several woody species. The amount of erosion is low in this reach, with some areas experiencing minor undercutting stabilized by large trees (Figure R3-5). Several sections of the reach are hardened by sackcrete structures and a large grade control structure is located at the downstream end of the reach.

Bed material is large throughout the reach, with a median grain size (D-50) of approximately 30 mm, measured using the conventional pebble-count technique (see Section 3.4.6).

Geomorphic Reach E

Upstream from Linda Court and downstream from the confluence with Ross Creek is Geomorphic Reach E. This channel reach has banks that are actively being eroded and are widening, leaving small roots exposed along the banks (Figure R3-6). Bed material in Geomorphic Reach E is smaller than that measured downstream, with a median grain size (D-50) of approximately 14 mm.

Geomorphic Reach F

Geomorphic Reach F is East Ross Creek, one of the major tributaries to Ross Creek. East Ross Creek joins Ross Creek upstream from Blossom Hill Road, and is underground for a portion of its length near the confluence. East Ross Creek has been modified for flood control purposes in the downstream-most sections (cross-section ER-1). However, the upstream, headwater portions of East Ross Creek remain in an unmodified form (Figure R3-7). The cross sections located on East Ross Creek progress from "high" erosion to "low" erosion in the upstream direction.

3.5.2.10 Discussion

Range of values

The project team compared the values of the data collected for Ross Creek to data recorded from other streams, including Thompson and San Tomas Creeks. Some parameters, such as slope and width-to-depth ratio, appear broadly typical of other stream systems in the region that are affected by hydromodification.

Width-to-depth ratios recorded in Ross Creek are typically in the range of three to four, compared to ratios of two to five reported for Thompson Creek and two to ten for San Tomas Creek. The range of width-to-depth ratio values is quite narrow for Ross Creek, with a standard deviation of only 0.40 for the mean value of 3.9, when excluding ER-3. This is due to the highly modified nature of most of Ross Creek.

Channelization and channel maintenance activities

The activity of constructing channels into a straightened planform with trapezoidal channel geometry affects the stability of these channels. Much of Ross Creek was initially channelized in the late-1950s for flood control purposes. One of the reasons Ross Creek was chosen as a test watershed was to test the HMP assessment method in these highly modified channels. Discussed below are some of the more pertinent ideas regarding channelization and Ross Creek that lend a framework for our results.

- Typical geomorphic responses associated with channelization, especially where channel straightening has occurred, are changes in channel morphology via re-meandering processes, incision, downstream aggradation, and bank failure. It is therefore extremely difficult to discern a specific cause of incision and general channel instability in the lower reaches of Ross Creek, rather it is likely a combination of factors that include major channel modification in the 1950s and increasing urbanization and runoff in the watershed since channelization.

- Distinct zones of channel bank instability occur along Ross Creek at intersections with the former 'natural' channel. One potential effect of straightening a meandering channel is creating a zone of weakness where the new channel intersects the former channel. The project team observed several of these channel intersections in the field in reaches that were experiencing large-scale bank failure.

- Channelization indirectly affects channel bank stability in similar ways as urbanization by the removal of bank and bank-top vegetation. This is compounded at Ross Creek by the construction of maintenance roads on one or both sides of the channel and regular vegetation maintenance activities, such as mowing.

3.5.3 San Tomas Creek (Upper)

3.5.3.1 Geomorphic reaches

Geomorphic reaches were not defined for San Tomas Creek because the number of cross-sections surveyed for the project was too small to fully characterize the stream longitudinally, given the diversity of observed conditions. The San Tomas Creek watershed is fairly diverse in terms of channel form, erosion pattern, and vegetation characteristics compared to Ross Creek, where geomorphic reaches were adequately defined and described by only eleven cross-sections. In general, the surveyed portion of San Tomas Creek had less engineered flood control structures than are present in Ross Creek. Also, not all of San Tomas Creek was surveyed due to access limitations.

3.5.3.2 Hydrographic segments

Hydrographic segments were also not defined for San Tomas Creek for similar reasons as Ross Creek.

3.5.3.3 Field Reconnaissance

Not all sections of the San Tomas Creek watershed were surveyed due to limited access in areas of private property. The following observations are among those made during reconnaissance visits in August 2003:

- Definition of geomorphic reaches in San Tomas Creek was not conducted; as the creek often changes physical characteristics (channel geometry, bank vegetation, and presence of dry-season flow) with each culvert/road crossing.

- San Tomas Creek flows into a concrete-lined channel downstream from McCoy Avenue and remains hardened to the Bay. Upstream from McCoy Avenue, San Tomas Creek is either an earthen trapezoid channel or an unmodified, 'natural' channel.

- Most of San Tomas Creek was dry during the August 2003 visits. Patchy areas of creek that sustained flow during the dry season were found in the lower sections of the watershed, where urban nuisance flows from outfalls were the likely source.

3.5.3.4 Channel geometry and channel slope

Several channel geometry parameters were calculated from the cross-section survey data for San Tomas Creek, including top-of-bank (TOB) widths, average depths from TOB, and width-to-depth ratios. Top-of-bank was used as a reference for width and depth measurements because bankfull dimensions, which are geomorphically more meaningful than TOB dimensions in self-formed alluvial channels, were not always present in modified and/or eroded sections of San Tomas Creek. The top of bank for each cross-section was identified in the field where possible, and interpreted from plotted cross-sections when field data did not explicitly designate a TOB. The top of bank can roughly be defined as the uppermost significant break toward a steeper slope

that the field crew encountered when conducting the cross-section surveys. Channel geometry and local channel slope measurements are summarized in Table S3-1.

Top-of-bank widths at cross-sections along San Tomas Creek range from approximately 12 to 38 feet. Cross-sections ST-1 and ST-2, which appear to have been widened and deepened for flood control purposes, have the widest channels, with TOB widths greater than 30 feet. Channel depths from TOB range from 2.0 to 11.4 feet at the surveyed cross-sections for San Tomas Creek. The modified cross-sections ST-1 and ST-2 also had the deepest channels. However, cross-sections ST-3 and ST-4, which have also been modified into straight, trapezoidal channels had shallow channel depths from TOB, of about 2 to 3 feet. This is because we designated the TOB as the bench that has formed closer to the bottom of the channel instead of the top of the levee. Top-of-bank dimensions were not calculated for cross-section ST-12 because TOB was not distinguishable. Cross-section ST-12 is the uppermost site surveyed and represents a steep, headwater channel within a canyon.

Width-to-depth ratio (top of channel width divided by mean depth) is a channel geometry parameter that can be used to compare different reaches within the same stream system to evaluate incision and channel widening. Typically, a high width-to-depth ratio represents a wide and shallow channel and a low width-to-depth ratio represents a narrow and deep channel. What constitutes a "high" value compared to a "low" value is dependent on the characteristics of the studied watershed and will vary by region. Width-to-depth ratios for San Tomas Creek vary from 3.1 up to 10.7.

Local channel slopes range from 0.0008 to 0.085 (ft/ft), with channel slopes generally increasing in the up-stream direction. Most measurements of local channel slope were conducted using bed surface elevations following the channel thalweg. Some water surface slopes were measured where there was flow.

3.5.3.5 Bank material

The banks in San Tomas Creek are primarily composed of three different types of material:

- Gravel and cobbles in a sandy, silty, matrix (alluvial deposits)
- Poorly consolidated sand and silt (colluvium or non-gravel bar deposits)
- Moderately-well consolidated sand, silt, and clay with gravel (sometimes engineered fill)

All but three cross-sections had banks composed primarily of alluvial deposits, with varying amounts of gravel and cobbles (ST-1, ST-2, ST-5, ST-7, ST-8, ST-9, ST-10, ST-11, and ST-12). Cross-sections ST-1 and ST-2 had several feet of engineered fill on top of the alluvial deposits. The three remaining cross-sections (ST-3, ST-4, and ST-6) had banks composed of poorly consolidated sand and silt, with little gravel and no cobbles.

3.5.3.6 Bed material

Table S3-2 includes results of the San Tomas Creek bed characterization study. D-50 values ranged from 13 to 32 millimeters, significantly higher than Ross Creek values. D-50 sizes increase upstream. There is some difficulty in directly comparing D-50 values, as the upper cross-sections were sampled using the pebble count method rather than by bed core. The bed material at cross-section ST-12 was not measured because the D-50 was assumed to be too large (cobbles and boulders) for most flows to transport.

3.5.3.7 Vegetation

San Tomas Creek has diverse vegetation characteristics, ranging from only grasses and shrubs to a full, dense riparian corridor. The type and density of bank vegetation often was dependent on location within the watershed, with the upper watershed cross-sections supporting more woody and dense vegetation on channel banks and floodplain areas than the lower cross-sections, which generally supported grasses and shrubs with some scattered trees. Cross-sections ST-5, ST-6, ST-7, ST-8, ST-9, ST-10 and ST-11 supported vines in conjunction with trees and grasses.

3.5.3.8 Erosion rankings

In total, 7 of the 12 cross-sections were assigned a "medium" or "high" erosion ranking, with one additional cross-section bordering between "medium" and "high." Cross-sections with "medium" or "high" erosion rankings are located throughout the San Tomas Creek watershed, with the exception of the headwaters and locations where the creek has been modified for flood control purposes.

Cross-sections where the erosion ranking was designated "low" typically had banks that were not eroded, eroded but now in a stable configuration, or eroding but at a typical rate and scale of natural channels. Four out of 12 cross-sections were designated as having "low" erosion. Two of these cross-sections (ST-3 and ST-4) are located in a section of San Tomas Creek that has been modified into an earthen trapezoid. These cross-sections show signs of former incision, but current, active erosion is low. Also, cross-sections ST-11 and ST-12, which are located in the headwaters of San Tomas Creek, were classified as "low-erosion" sites.

3.5.3.9 Cross-section characteristics

Twelve cross-sections were studied in detail in the San Tomas subwatershed. The cross-sections represent portions of the San Tomas subwatershed, rather than the entire subwatershed. Some sections of San Tomas Creek were not studied due to inaccessibility. Although no continuous geomorphic reaches were established for San Tomas Creek, the cross-sections can be grouped in some instances based on similar geomorphic characteristics and/or geomorphic history.

Cross-sections ST-1 and ST-2

Cross-sections ST-1 and ST-2 are located just upstream from McCoy Avenue and where San Tomas Creek first becomes hardened for a substantial length. This section of creek has been

channelized in the past and is currently unstable. At both cross-sections the channel banks are actively eroding, and incision is noted in the thalweg at cross-section ST-1.

Near cross-section ST-2, the left channel bank around a rock gabion structure in the channel has eroded laterally approximately four to five feet. The gabion structure was likely initially installed for toe protection; another suggestion that bank widening has been and continues to be a problem in this section of San Tomas Creek.

Cross-sections ST-3 and ST-4

It is unclear if both cross-sections ST-3 and ST-4 have been channelized in the past based on field evidence alone, although ST-3 most likely has undergone substantial modification and possible straightening. This section of creek has several grade control structures that maintain a fairly gentle slope. A maintenance road is located on the top of the channel bank.

Some minor bank undercutting was evident at cross-section ST-3, but it was deemed minimal. Both cross-sections, ST-3 and ST-4, were ranked as "low-erosion" sites (Figure S3-2). Cross-section ST-4 might be experiencing aggradation due to the location of a grade control structure approximately 500 feet downstream.

Cross-section ST-5

Cross-section ST-5 is located just upstream from Pollard Road. This section of creek appears highly unstable, with active bank erosion and possible incision. There are tree roots exposed along the channel banks. The channel banks are steep although vegetated at the top of the bank and are without maintenance roads. Cross-section ST-5 was ranked as a "high-erosion" site.

Cross-section ST-6

The field crew located cross-section ST-6 at a section of San Tomas that was experiencing active bank erosion. Cross-section ST-6 was ranked as a "medium-erosion" site. This cross-section has steep banks that are eroding at two different levels, a) the bottom layer on the right bank composed of silty and sandy clay with organics is being actively undercut and b) the upper layer on both banks composed of non-consolidated silty and sandy clay has several active failures (bank slumping).

Cross-section ST-7

Cross-section ST-7 is located just downstream from Saratoga Road (Route 9) in a densely vegetated section of San Tomas Creek. Recent incision has occurred in this section of San Tomas Creek of about six to eight inches, which is seen best upstream from the cross-section. Active bank erosion is also occurring at cross-section ST-7 in the form of undercutting and shearing. A retaining structure failed downstream from the cross-section due to a combination of incision and bank widening. Cross-section ST-7 was ranked as a "high-erosion" site.

Cross-sections ST-8, ST-9, and ST-10

Cross-sections ST-8, ST-9, and ST-10 are all located in close proximity to each other upstream from Saratoga Road and near Bantier Way. They are ranked as either "medium-erosion" or "high-crosion" sites. Cross-section ST-8 is located downstream from a large stormwater outfall, whereas cross-sections ST-9 and ST-10 are located upstream from the outfall where the watershed is relatively un-urbanized, limited to large-acreage residential uses

Several faults and geologic contacts are located in this section of San Tomas Creek and represent a significant transition between the tectonically active, headwaters and the alluvial plain (Santa Clara County Geologic Map, 1974). Based on field evidence, it is likely that cross-sections ST-9 and ST-10 have been influenced by a series of historic debris flows and that some of the incision and bank instability noted at these sites are due to these naturally recurrent episodic events.

Cross-sections ST-11 and ST-12

The upstream-most cross-sections of ST-11 and ST-12 are similar in that both are relatively stable and not actively eroding more than what is typical for headwater streams. The bed material carried in the upper watershed of San Tomas is quite large, cobbles at ST-11 and boulders at ST-12 (Figure S3-3). Even less urbanization has occurred upstream from these cross-sections compared to ST-9 and ST-10.

3.5.3.10 Discussion

Range of values

The project team compared the values of the data collected for San Tomas Creek to data recorded from other streams, including Thompson and Ross Creeks. Some parameters, such as slope and width-to-depth ratio, appear broadly typical of other stream systems in the region that are affected by hydromodification.

Width-to-depth ratios recorded in San Tomas Creek have a fairly broad range from two to ten, compared to three to four at Ross Creek and two to five at Thompson Creek. This reflects the diverse channel morphologies the project team observed in the field.

Dry-season flows

Due to our fieldwork being conducted in the summer and fall months, the field crew was able to observe and evaluate dry season flows along San Tomas Creek. During the survey, which was conducted in September 2003, four of the twelve cross-sections were flowing; ST-2, ST-4, ST-5, and ST-7. The field crew could see where the water table was intercepting the channel bank at cross-section ST-2, but signs of water further upstream (ST-4, ST-5 and ST-7) likely indicate dry season nuisance flows entering the system from stormwater outfalls rather than ground water sources because no seeps were observed and the segments of ponded or flowing water were discontinuous.

3.6 Subwatershed comparison

Field geomorphic surveys are an essential component of developing, testing, and ultimately implementing a set of methods and effective control measures that address hydromodification.

By comparing the geomorphic characteristics of three creeks investigated for the HMP study (Ross, San Tomas and Thompson Creeks), one can better understand what influences channel stability in different physical settings. One can also compare parameters thought to be critical in defining stability (or the ability of channel to resist erosion), such as bed-material size, bank-material composition, vegetation, and others to see if patterns are detected in the different regions of the Basin.

- The single physical parameter that best reflects the stability of a cross-section in these streams is the width-to-depth ratio (top of bank width divided by the average depth from top of bank). Stable channels, or "low-erosion" sites, tended to have larger width-to-depth ratios than unstable channels, ranging from three to ten with an average of approximately 5.5. Unstable channels, or "high-erosion" sites, however, exhibited a much narrower range of width-to-depth ratios of between one and five, with an average of 3.2. Only two unstable cross-sections had width-to-depth values greater than 3.9. Channel width values in isolation were not substantially different between stable and unstable channels.[12]

- In general, similar modes of bank failure were observed in the three test watersheds even though they are located in different geologic/geomorphic regions of the basin. The primary mode of failure being shear erosion at the toe of the bank and bed, which results in incision and over-steepened bank. An array of secondary factors, such as vegetation or lack thereof, can play important roles in bank failure in conjunction with erosion resulting from shear stress. It is meaningful to note that some of the secondary modes of bank failure were more prominent than others in specific watersheds due to differing environmental settings.

    - The key secondary modes of bank failure in the Thompson Creek watershed are a lack of bank and top of bank vegetation in highly urbanized and/or space-constrained reaches and rapid drawdown of flows, which is a product of the flashiness of the hydrograph.

    - Secondary modes of bank failure in the Ross Creek watershed are a result of channel modification in the lower reaches of the creek. These secondary modes

[12] Hupp and Simon (1991) present a model of channel evolution that describes how channels respond to disturbances, such as urbanization. In their model, incision is the initial mode of channel response followed by channel widening. It is possible that although channel widths do not differ much among the three subwatersheds at present, future widening could occur in channels that are currently experiencing incision.

of failure are a) a lack of bank and top of bank vegetation (especially woody species) due to the construction of maintenance roads that run parallel to the channel and b) contacts between the former channel and the engineered channel that create zones of weakness in channel banks.

- San Tomas Creek also experiences bank failure due to a lack of bank vegetation, but is unique amongst the three watersheds because its headwaters are located in the Santa Cruz mountains; an especially tectonically active part of the Basin. The upper portions of San Tomas Creek are periodically subject to debris flows and landslides, both of which affect channel bank stability.

4 Hydrology

This chapter describes the project team's hydrologic modeling of the Thompson and Ross and San Tomas Creek watersheds. Modeling was conducted for the hydromodification assessment and planning process.

4.1 Modeling Approach

The project team modeled creek flows under pre-urban, existing, and future land use conditions. The watershed models convert rainfall input sequences to estimated stream flow rates at various selected points throughout the project watersheds. The model rainfall input consists of simulated "design storms" and continuous rainfall records. Design storms were developed for various rainfall magnitudes, intended to simulate major flood-causing events, such as the 2-, 10- or 100- year rainstorms. We refer to this type of modeling as "event-based" because only a single rainstorm event (ranging from perhaps 3 hours to 24 hours) is simulated. Alternatively, we may input actual measured rainfall from a nearby gage over a long period of time. This is referred to as "continuous simulation."

Within event-based and continuous simulations, the model incorporates information about the watershed characteristics (topography, soils, vegetation, land use, urbanization, etc.) to estimate how much rainfall is held in the watershed ("losses", including infiltration to the soil, trapping on vegetation or shallow depressions, etc.), and how much precipitation results in surface runoff, eventually reaching stream channels.

The project team chose to model the Thompson and Ross and Upper San Tomas Creek watersheds using the U.S. Army Corps of Engineers' Hydrologic Engineering Center - Hydrologic Modeling System (HEC-HMS) rainfall-runoff model. The U.S. Army Corps of Engineers developed HEC-HMS to supersede the HEC-1 Flood Hydrograph Package. Unlike HEC-1, HEC-HMS allows continuous hydrograph simulation over long periods of time in addition to event-based analysis.

Event-based modeling is useful because it provides a simple method for comparing hydrograph results under different land use conditions for statistically relevant design storms. In addition, event-based modeling is a convenient and commonly accepted approach for evaluating flood risk and design alternatives. Continuous modeling, however, allows for continuous accounting of soil moisture and infiltration and other losses for an extended time period. Therefore, continuous modeling is preferable to event-based modeling when trying to identify the hydromodification effects of development on small, frequent flows and to evaluate their impacts on stream stability.

4.2 HEC-HMS Model

The following sections describe the methods and data sources used to generate input for the HEC-HMS models. The modeling approach generally followed methods and procedures for HEC-1 modeling outlined in Santa Clara Valley Water District's *Hydrology Procedures* (SCVWD, 1998) as noted below. A previous Thompson Creek hydrology study conducted by Nolte Associates (2000) and a Ross Creek study described in SCVWD's *Hydrology*

Procedures also provide background information and estimated flow peaks for model comparison.

4.2.1 Drainage Area Delineation

Project watersheds were subdivided into smaller subwatersheds or catchments to provide a detailed assessment. Using GIS data, the project team delineated catchments associated with storm drain outfalls, storm drain flow direction, and topographic data. Catchments were further delineated to reflect land-use patterns. To the extent possible, individual drainage areas were delineated to separate developed (urban) and undeveloped (rural) areas, as many model parameters are derived from a drainage area's weighted average characteristics and are specific to degree of urbanization. Catchments were consolidated for the pre-urbanization land use scenarios, for which fewer urban areas were present. Figures 4-1 through 4-3 show drainage area delineation of the study watersheds. Tables 4-1 through 4-2s provide catchment size.

4.2.2 Drainage Area Characteristics

The project team identified land cover characteristics and soil types for the study watersheds based on the project's GIS database. The project team overlaid the drainage area delineations on those data to derive soil and land cover characteristics used in modeling each drainage area (Figures 4-4 through 4-6).

Existing hydrologic conditions were modeled using detailed soils and land use GIS data from SCVWD. The land use data were then modified to model hydrologic conditions for future and past (pre-urban) conditions, since GIS data were not available for these scenarios. For future conditions, the percentage of impervious land for each subwatershed under current conditions was increased based on future build-out percent impervious information from the City of San Jose General Plan 2020 (Mattern, 2003). All other land uses for each subwatershed were then decreased in proportion to the increase in impervious area.

The project team reviewed historic aerial photographs and USGS topographic maps to characterize pre-urban land use conditions. These sources provided a representation of the pre-urban distribution of agricultural and woodland/grassland areas for each subwatershed, which was then converted into model input parameters.

4.2.3 Excess Rainfall

HEC-HMS uses soil infiltration rate estimates and other losses described below to calculate excess precipitation that contributes to stormwater runoff. The event-based HEC-HMS model uses the Soil Conservation Service (SCS) method to estimate losses, and the continuous simulation uses the Soil Moisture Accounting (SMA) method (unique to HEC-HMS).

SCS Method (Event-based Model, used in Thompson Creek study only)

The SCS method uses a watershed runoff curve number (CN) to represent runoff potential for various soil types and land coverages. To determine CNs, the project team first assigned a soil infiltration potential to each identified soil type by using the NRCS Soil Hydrologic Group designation (A, B, C, or D with soil type A having the highest infiltration potential). Using guidance from *Hydrology Procedures* (SCVWD, 1998) and assuming an average antecedent moisture condition (AMC II), the team then assigned a CN value for each land coverage and soil group combination. Low CNs reflect a high infiltration potential. In regions where the land cover was identified as structures or paved surfaces, the CN value was set to 98, regardless of soil type. The project team then generated a CN for each drainage area using an area-weighted average of CN values present. The project team also used the SCS method to estimate initial abstraction losses as a function of CN (SCVWD, 1998). These parameters are summarized in Table 4-3.

Soil Moisture Accounting Method (Continuous Model, used for Thompson Creek, Ross Creek and Upper San Tomas Creek)

The SMA method provides a more complex method for evaluating rainfall runoff processes in a watershed. In this approach, actual measured rainfall over an extended time period is used as input. Losses are computed on a continuous basis, and include evapotranspiration, surface depression storage, and infiltration. The continuous model is designed to model the dynamic effect of soil infiltration and other losses on storm runoff over the course of a long-term rainfall record. Parameters to compute these losses include climatic data, land use conditions, vegetation cover, and soils data. The simplified conceptual schematic of Figure 4-10 illustrates the SMA model:

For each computational time step in the model, HEC-HMS calculates storage in each of the loss categories shown in the schematic, which allows for a continuous accounting of losses and runoff over a long time series. For infiltration, the model initially assumes that water enters the soil at the maximum infiltration rate and percolates out of the soil at the maximum percolation rate. Once the soil layer becomes saturated, the infiltration rate is reduced to the percolation rate. SMA parameter estimation is described in section 4.3.

4.2.4 Hydrograph Generation

Initially, the model determines how much incident rainfall is held in the watershed (losses), and how much will appear as runoff. That which appears as runoff is referred to as "excess precipitation." The model then determines the time distribution of this watershed-wide excess precipitation, as it flows across the land surface or in shallow "interflow," eventually reaching culverts or small drainage channels, and finally the main stream channel at the various flow computation points of interest. The resulting time distribution of runoff at a given location is referred to as "hydrograph."

HEC-HMS offers a variety of methods for transforming excess precipitation from any given storm into a runoff hydrograph for each model drainage area. SCVWD's *Hydrology*

Procedures (1998) recommends and provides guidance for using Clark's synthetic unit hydrograph method in HEC-1. Clark's method requires two inputs: time of concentration (T_c) and a storage coefficient (R). We used the District methods to estimate these two drainage area parameters (Tables 4-4 through 4-13).

The procedure described in *Hydrology Procedures* (SCVWD, 1998) requires further sub-division of urbanized drainage areas into pervious and impervious areas, and those areas that drain to storm drains. Runoff passing through storm drains is then routed through storage/discharge relationships based on the "Modified Puls" study conducted by the District for Santa Clara Valley storm drains (SCVWD, 1998). Under the SCVWD procedure, hydrographs from pervious and impervious urban areas within a drainage area are routed through the storm drain system, and then combined with the hydrograph for the portion of the drainage area (if any) that does not enter the storm drain system. This routing and combination routine is internal to each drainage area, in contrast to reach routing between drainage areas. As the modeling progressed from Thompson Creek to Ross Creek, it was determined that the urban storage component was unnecessary. Therefore storage components within the HEC-HMS model were only developed for Thompson Creek (Table 4-14)

4.2.5 Reach Routing

HEC-HMS provides a variety of reach routing methods to translate the hydrograph from one drainage area downstream to a point where it can be combined with another drainage-area hydrograph. The project team chose to use the Muskingum method, which uses basic channel (or culvert) dimensions and characteristics to estimate hydrograph translation and attenuation over the routing reach. For existing and future conditions, a combination of surveyed cross-sections, available storm drain data, and information from the District was used to characterize channel dimensions and characteristics for reach routing. For the pre-urbanization scenarios, estimations were established for channel characteristics and flow paths using historic aerial photos. Reach routing parameters are summarized in Tables 4-15 through 4-18.

4.2.6 Precipitation

Event-based Model

Synthetic rainstorms were developed to simulate various flood events. Rainfall depths for the 2-, 5-, 10-, 25-, 50-, and 100-year storms (24 hour duration) were estimated using the "return period-duration-specific" (TDS) regional equation provided in *Hydrology Procedures* (SCVWD, 1998). The TDS equation provides a rainfall depth for each storm based on a site's mean annual precipitation (MAP) and coefficients developed by SCVWD from long-term rainfall records. The project team estimated rainfall depth for each design storm for each model drainage area based on the distribution of MAP. Figure 4-11 shows MAP in the vicinity of the study watersheds.

The "balanced hyetograph" routine in HEC-HMS generated storm hyetographs for each return period based on the rainfall depths described above. A storm hyetograph describes the

time distribution of the rainfall intensity over the duration of the storm. A balanced hyetograph provides a distribution of the rainfall that maintains the same recurrence event for all time intervals within the storm; thus, a "balanced" 100-year, 24-hour hyetograph will include 100-year rainfall rates for the 0.5-hour storm, the 1-hour storm, and the 3-hour storm imbedded within the total rainfall distribution. This method is useful in simulating runoff in a multi-drainage area watershed model with drainage areas of different sizes, as in this study, since it insures that catchment areas of different sizes will all experience the same recurrence interval storm. This occurs because watersheds of different sizes respond differently to different rainfall distributions: a smaller watershed will produce higher flow rates from a short-duration, intense rainstorm, while a larger watershed may experience higher peak flow rates from a longer duration, less intense storm. The use of the balanced hyetograph allows the inclusion of both shorter duration peaks and longer duration characteristics all within the same design storm.

Continuous Model

The HEC-HMS continuous simulation was run using National Climatic Data Center (NCDC) continuous, hourly rainfall data from gage station ID 047821, the "City of San Jose" gage for a 50-year period (records for summer of 1948 through summer of 1999 were used within the model). This gage corresponds to SCVWD station 6086, which was moved in 1986 to a nearby location known as Station 6131 (San Jose Airport). The data collection method was also changed at that time, from a 0.01-inch recording increment to a 0.10-inch increment. This change is reflected in the rainfall record, in that the smallest rainfall event recorded after 1986 is 0.10 inch. Using a process similar to that described for the event-based model, the project team scaled the continuous rainfall record for each drainage area based on drainage-area Mean Annual Precipitation (MAP).

Figure 4-11 highlights the location of the City of San Jose precipitation gage. The City of San Jose gage, although located outside the study area, has recorded a significantly longer period of precipitation than have other gages in the area. It is recognized that measured rainfall at the Airport, scaled by MAP, is only an estimate of rainfall distributed across the study watersheds. Actual rainfall rates vary spatially, and intense rainfall rates (resulting from individual convective cells within a rainstorm) often occur over one area, but may miss another area nearby. Thus, while measured rainfall at the Airport gage represents a valuable estimate of rainfall for the project watersheds, variations during any individual storm are likely.

Distributed Precipitation

For design storms and for continuous precipitation records, the project team used GIS to divide each study watershed into categories according to MAP ranges. Precipitation input was then adjusted by category according to estimated rainfall depth. MAP categories were assigned to each model catchment, based on average MAP, and the corresponding adjusted precipitation was linked to each catchment.

4.3 Parameter Estimation Process

To simulate the watershed response to a rainfall event, a variety of parameters must be estimated in the hydrologic model. These estimated parameters affect the size and shape of the storm hydrograph predicted by the model compared to what may result from any individual actual storm. Whenever possible, modelers compare model results to recorded concurrent rainfall and flow data to calibrate the model by adjusting various parameters to reproduce the actual flow resulting from measured rainfall. The project team generally calibrated the models by adjusting SMA parameters and unit hydrograph values.

Initial estimations of SMA parameters were developed in accord with the methodology outlined in Table 4-19. Model calibration refined the parameters within acceptable parameter ranges. A preliminary calibration was performed for the Thompson Creek watershed, which will be refined as additional stream flow measurements are obtained. Historic stream gage data was not available to calibrate the Thompson Creek HEC-HMS model to actual flows in the Lower Silver - Thompson Creek system. However, the SCVWD stream gage on Thompson Creek at Quimby Road has been functional since January 2003, and additional flow data for Thompson Creek is currently being collected as part of a separate project. These sources will provide useful calibration data in the future. Until more data becomes available and the Thompson Creek model is fully calibrated, a level of caution should be extended if model results are to be used in a broader context.

In the absence of calibration data, the project team compared model results to other available peak flow estimates in order to verify the "reasonableness" of model results. Peak flow estimates used in this comparison include those from SCVWD 1998 ("flood quantiles" or regional regression equations), Nolte 2000, and FEMA 1986, as described below. In addition, Thompson Creek peak flows were estimated for a December 2002 storm event using high water marks. Rainfall records from that storm were used in the continuous model. Results from the model were then compared to the estimated peak flows, and used to adjust the model parameters. More recently, measured flow data for the period January-May 2003 has become available. Additional collection of data will allow for a more detailed calibration of model parameters.

A continuous record of stream gage data was not available to calibrate the continuous simulation of the HEC-HMS model to actual flows in the Lower Silver - Thompson Creek system. The project team therefore used the results of the event-based model to verify that the continuous model produces reasonable results. To obtain comparable "peak flows" from the continuous model, we initially ran the model to provide 50 years of estimated flow records. The peak flow rate for each year in the record was then identified. Estimates of the 2-, 5-, 10-, 25-, 50- and 100-year return period flow rates were obtained by performing a statistical flood frequency analysis of the ordered flood peaks, using the same method that is used for actual gaging station flow records. This analysis was conducted with the model parameterized for the existing land use condition. Peak flows were then compared to those estimated by the event-based model for the same return periods and land use scenario. Initial estimates of SMA model parameters, specifically the infiltration parameters, were then adjusted to produce peak flow rates that were in reasonable agreement with the event-based model.

Given the limitation of available calibration data, the Thompson Creek continuous model input parameter refinement process has primarily focused on matching observed and simulated flow volumes. The process, to date, has been a model "verification" as opposed to "calibration." Future calibration of the Thompson Creek model may include flow volume, peak flow rates, and hydrograph shape as calibration criteria.

4.4 Methodology for Hydrologic Analysis

4.4.1 Flow Frequency Analysis

Annual Flow Frequency

Recurrence intervals of simulation results are represented by ranked annual peak flows based on USGS Bulletin 17B guidelines for distributions fit to a log-Pearson Type III frequency distribution. Under the 17B guidelines, the distribution was weighted with generalized skew coefficients applicable to the study area. The USGS model PEAKFQ was utilized to calculate the skew and the subsequent recurrence intervals for the data sets.

All-Event Flow Frequency

Traditional hydrologic analysis has focused on low frequency/high flow events, as these tend to be of primary interest for flood hazard prediction. Typically, storms between the 10 and 100-year event are simulated. However, one of the key changes that occur as a result of hydromodification is the dramatic increase in the frequency of runoff from smaller storms. In pre-urban conditions, many of these rainstorms would produce no runoff at all, with all of the precipitation trapped on vegetation, in shallow depressions, and infiltrating into the soil. The large areas of impervious surface associated with urbanization, connected directly with storm drain culverts, results in more frequent runoff, and considerably faster travel times than in the pre-urban state. Frequent small storms, with return periods of 2 years or less, are influential events for long-term sediment movement. Recurrence intervals were calculated for all flows events obtained from the continuous simulations using individual event separation, as opposed to annual peak flows only. We refer to this methodology as "all-event flow frequency." Flow frequency relationships calculated using the annual peak methodology described above do not capture the smaller, frequent storm events of interest to the project team.

For this analysis, the peak flow rate from each individual storm was computed, stored and tabulated from the continuous simulation record for the three land use scenarios (pre-urban, existing, and future land use conditions). To extract events from model output, the beginning and end of flow events were marked by the rise and fall of flow above a threshold level. Peak flows from each event were ranked in descending order and assigned a monthly return period, T, as follows:

$$T = (N + 0.2) / (M - 0.4)$$

such that N is the number of months simulated, and M is the event rank (adopted from James *et al.*, 2002). The return periods are the probability that particular events will be observed

within an average month. Note that this technique does not account for seasonality of precipitation events, or probability of an event happening during a "wet" vs. "dry" month. In that context, an event assigned a three-month return period might be more appropriately referred to as an event that occurs four times per year on average.

4.4.2 Regional Regression Flow Estimates

The project team compared peak flows generated by the model to peak flows predicted by the regional regression equations provided by SCVWD (1998). The regional equations provide a method for estimating peak flows for a variety of return periods, based on contributing sub-watershed area, MAP and other parameters (SCVWD, 1998). The equations are derived from analysis of long-term historical stream gage records for the region, and are generally assumed to represent undeveloped watershed conditions.

It should be noted that the regional regression equations, the event-based modeling approach, and the continuous model represent three completely independent methods of estimating the larger magnitude flood events.

- The regional regression equations are based on actual gaged flow records for undeveloped watersheds, then "non-dimensionalized," based on annual precipitation and watershed area.
- The event-based model uses District-selected parameters, with a "design" rainstorm. The rainstorm volume is derived from observed frequency curves, but the temporal distribution (hyetograph) is "artificial", based on experience.
- The continuous model approach attempts to simulate a gaging station flow record (using measured rainfall only). The simulated flow record is then statistically analyzed to estimate flood frequency.

The inclusion of all three of these methods provides some measure of the uncertainty inherent in hydrologic analysis.

4.5 Thompson Creek Watershed

4.5.1 Continuous Model Results

This section presents data from the continuous model for pre-urban, existing and future land use conditions. Due to the large amount of data generated by the model for each land use scenario (50 years of flow estimates at half-hour intervals for multiple locations), summary results for junctions J-5 and J-12 are presented in this section to demonstrate representative model results for the upper and lower portions of the Thompson Creek watershed, respectively.

Figures 4-12 and 4-13 show the results of a flood-frequency analysis of continuous model results for the three land use conditions at the two representative locations. Figure 4-13 shows results for the lower portion of the watershed, reflecting aggregated hydromodification effects of development throughout the watershed. These results demonstrate the fact that the

watershed is largely built-out in the lower reaches, so that the projected hydromodification between the current and future conditions is much less than what has occurred from the 1960's ("pre-urban" condition) to the present. The upper reaches of the watershed are partially developed at present, although additional development is planned in those locations for the future. Potential impacts due to increased imperviousness in the upper reaches may be significant.

A comparison of 10-year peak flows at J-12 demonstrates an important aspect of hydromodification. As shown in Figure 4-13, the 10-year peak flow for the pre-urban condition (approximately 1500 cfs) now occurs approximately every 2.5 years under existing conditions, according to the continuous model results. This represents a dramatic increase in the frequency of flows of this magnitude, which has implications for stream stability, as discussed in another chapter. The increased flow frequency is less dramatic between current land use conditions and future, but is still potentially significant.

Model results for J-5 (Figure 4-12) show a less dramatic difference between pre-urban and existing conditions, since the upper part of the watershed is not yet built out. However, a similar comparison peak flows show that the 10-year peak flow under pre-urban conditions now occurs approximately every six years under current conditions, according to the model results.

Recurrence intervals calculated using annual peaks from the continuous model are compared with the all-event flow frequency in Figures 4-14 and 4-15. Estimated peak discharges from the two methods compare favorably for return intervals of five to 10 years and above. However, peak discharges calculated using annual peaks do not capture the smaller, frequent storm events of interest for this project.

As seen in Figures 4-16 and 4-17, the percent increase in discharge, for a given return period, is relatively small when comparing existing to future conditions. The change from pre-urban to existing or future conditions is significantly larger, particularly for small, more frequent storm events. For the events with a very low return period (3 months or less), the percentage increase between pre-urban and existing (or future) is enormous. This demonstrates the important role that imperviousness has on runoff generation.

Although historic measurements of flow in Thompson Creek were not available, some flow data collected between January and May 2003 were made available for use in assessing initial model performance. The continuous model was compared with gaged streamflow for the period of January-April 2003. Balance Hydrologics provided flow data for Thompson Creek near Quimby Road and Yerba Buena Creek near the confluence with Thompson Creek. Measured precipitation for this same period was obtained from the San Jose Airport raingage, and scaled by MAP, was used as the model input (Figure 4-18).

The primary form of model assessment was a comparison with predicted and measured streamflow volume for both individual storm events, and the rainy season as a whole. This process indicates if the model is generally separating out rainfall runoff and losses within the watershed. Calibration of individual storm hydrographs and peak flow rates for small storms is difficult, due both to the complexities of the watershed response at that scale, and also, the

difficulty of accurately extrapolating the rainfall data from outside the watershed to the response at different watershed junctions. Therefore, the Project Team compared simulated volumes of flow, rather than individual hydrograph peaks, to measured volumes at the gages.

Measurements from the Quimby Road gage were compared with simulated flows at junction J-11 of the model. Discharges measured at the Yerba Buena gage were compared with the combined simulated flows from STO-8 and R-12. Figures 4-19 and 4-20 show cumulative volumes of flow as measured by the stream gages and as computed within HEC-HMS. Cumulative precipitation is also shown within the figures. Simulated discharge is consistent with measured discharge for Thompson Creek at Quimby Road. However, at the Yerba Buena site, model results over-predict discharge volumes. Overall, it is important to recognize that only about 6- to 10-percent of the rainfall in the watershed appears as stream runoff. This is a relatively small percentage, based on our experience in other watersheds throughout the region (where typically, 15- to 25- percent of the total rainfall appears as runoff). The majority of rainfall is held in the watershed, then later either lost via evapotranspiration, or flows to the Bay through subsurface layers as groundwater. The continuous model provides a good simulation of these overall watershed dynamics.

Individual event volumes were calculated for several flow events. Table 4-20 lists computed and measured volumes for events occurring January 9-10, February 16, February 25-27, and March 15-16, 2003. Figures 4-21 and 4-22 graphically present the volumetric comparisons. In general, measured discharge volumes for Thompson Creek at Quimby Road correspond to approximately 4-6% of the event precipitation volumes. The continuous model calculates flow volumes that compare reasonably well to the measured values. Measured streamflow at the Yerba Buena site is generally less than 1.5% of the precipitation volume. Simulated volumes at Yerba Buena compare well considering that the model is not yet calibrated.

Based on initial simulations using spring 2003 rainfall data, it appears that with the present parameterization, the continuous model tends to over-predict runoff and most peak flow rates. Therefore, after calibration, the simulated flows will most likely decrease. It is important to note however, that the relative differences between pre-urban, existing and future conditions are unlikely to change very much as a result of calibration.

4.5.2 Event-Based Model Results

The project team compared peak flows generated by the event-based model to peak flows predicted by the regional regression equations provided by SCVWD (1998). The regional equations provide a method for estimating peak flows for a variety of return periods, based on contributing sub-watershed area, MAP and other parameters (SCVWD, 1998). The equations are derived from analysis of long-term historical stream gage records for the region, and are generally assumed to represent undeveloped watershed conditions. Event-based results were also compared to flow estimates provided in Nolte 2000.

Figures 4-23 and 4-24 compare simulated event peak discharges as a function of yearly recurrence intervals at two HEC-HMS nodes (junctions J-5 and J-12). Discharges are shown for simulated pre-urban, existing, and future development scenarios plotted with discharges calculated from the regional equations. Estimates from Nolte 2000 are also shown for

comparison in Figure 4-24. The line representing the historical flood quantile (regional regression) shows good correlation with simulated pre-urban conditions. This relationship is reasonable as the regional equations were developed based on gaging station data from primarily undeveloped watersheds.

Figures 4-25 through 4-28 show additional event-based model results for past, existing and future land use conditions at each segment of Thompson Creek, and for the upper portion of Yerba Buena Creek (refer to Figure 4-1 for junction locations). Peak flows for a variety of return periods are plotted together for the three land use scenarios. This comparison demonstrates the effect of urbanization on peak flows across the spectrum of return periods, with the most dramatic impact being on the smaller, more frequent storms.

A summary of model results (peak flows and discharge volumes) is presented in Table 4-21, for relevant locations on Thompson Creek and Yerba Buena Creek. In addition, model results at J3 and J12 for selected return periods are presented in Figures 4-29 through 4-32.

4.6 Ross Creek Watershed

4.6.1 Continuous Model Results

Continuous discharge records from two Ross Creek stream gages were used to calibrate the HEC-HMS model. Gage #21 is located on Ross Creek at Blossom Hill; Gage #51 is located at Cherry Avenue. Gages #21 and #51 correspond to HEC-1 model nodes J-5 and J-1, respectively (Figure 4-2). Due to the large quantity of data generated in the Ross Creek model, this section will limit the results discussion to results from junctions J-5 and J-1. Junction J-5 represents the upper watershed, which is largely undeveloped under existing conditions. Junction J-1 is inclusive of the upper and lower watershed and includes the urbanized areas of the watershed.

Figures 4-33 and 4-34 show the results of a flood-frequency analysis of continuous model results for the three land use conditions at the two representative locations. The Ross Creek Watershed, similar to the Thompson Creek Watershed, is largely built-out in the lower reaches. The effect of watershed hydromodification is much more significant when comparing pre-urban/existing conditions than when comparing existing/future land use conditions.

Due to increased urbanization, and the resulting increase in impervious area, the frequency of flows at a given magnitude have increased. As illustrated in Figure 4-34, the 10-year pre-urban peak flood event was approximately 500 cfs. According to the model results, under existing conditions, a flow of that magnitude is expected to occur almost every year. A 500 cfs magnitude event is expected to occur slightly more frequently under future land use conditions. As seen in Figure 4-33, the 10-year pre-urban peak of 250 cfs occurs, on average, every 1.3 years under existing conditions. The increase in flow frequency is a significant influence on watershed stream stability.

Recurrence intervals calculated using annual peaks from the continuous model are compared with the all-event flow frequency in Figures 4-35 and 4-36. The two methods are

comparable for events larger than the 3-year return period. However, computation of an annual flow frequency does not capture the smaller, sub-year, events that are important for long-term stream stability.

Figures 4-37 and 4-38 present the percent increase in discharge, for a given return period, for the multiple land use scenarios. As expected, the change from pre-urban to future land use conditions represent the largest differences in peak flow. The hydrologic peak flow comparison resulting from changing existing land use to future conditions is relatively small.

The pre-urban and existing condition models for Ross Creek were calibrated against the observed data for two flow gages within the Ross Creek watershed. In the upper portion of the watershed, the flow record for Ross Creek Gage 21 at Blossom Hill Road was used to calibrate the model at junction J-5. Flow data was available from 1946 till 2003 for Gage 21, and the period from October 1950 through March 1952 was used for the pre-urban condition calibration of the model at J-5. The hydrograph results for the pre-urban condition calibration at J-5 are shown in Figure 4-39.

The flow record for Ross Creek Gage 51 at Cherry Avenue was used to calibrate the lower portion of the Ross Creek watershed model at junction J-1. Flow data was available for this gage from 1956 through 2003, and the period from October 1956 through May 1958 was used for the pre-urban condition calibration of the model at J-1. The hydrograph results for the pre-urban condition calibration at J-1 are shown in Figure 4-40.

For the existing condition model for Ross Creek, junctions J-5 and J-1 were both calibrated for the period from November 2000 through March 2003, using flow data from Gage 21 and Gage 51, respectively. The hydrograph results for the existing condition calibration at J-5 and J-1 and are shown in Figure 4-41 and Figure 4-42, respectively.

Tables 4-22A through 4-22D list the numerical results for each of the four calibration periods discussed above. The total volume for the flow gage record and the simulation are shown, as well as the total volume for the flow gage minus all flows less than or equal to 1 cfs. This analysis was added to account for the effects of baseflow in the flow gage record. The model results for total volume were compared with this decreased volume, so as to not over predict flow volume in the model, which did not simulate baseflow. The percent error from observed volume is presented the tables, and due to the variability of hydrologic modeling, a deviation of 20 percent is considered a strong correlation. The average discharge for each condition was included for comparison. The RMS error function value for the model results when compared to both the gage data and the adjusted gage data are presented in the tables.

The degree of correlation between the observed and simulated flows was measure using the peak-weighted root mean square (RMS) error objective function. This function is identical to the calibration objective function included in computer program HEC-1 (USACE, 1998). It compares all ordinates, squaring differences, and it weights the squared differences. The weight assigned to each ordinate is proportional to the magnitude of the ordinate. Ordinates greater than the mean of the observed hydrograph are assigned a weight greater than 1.00, and those smaller, a weight less than 1.00. The peak observed ordinate is assigned the maximum weight. The sum of the weighted, squared differences is divided by the number of

computed hydrograph ordinates; thus, yielding the mean squared error. Taking the square root yields the root mean squared error.

Therefore, this function is an implicit measure of comparison of the magnitudes of the peaks, volumes, and times of peak of the two hydrographs. The function is defined as follows:

$$Z = \sqrt{\frac{\sum_{t=1}^{n} (Q_0(t) - Q_S(t))^2 \frac{Q_0(t) + Q_A}{2Q_A}}{n}}$$

$$Q_A = \frac{1}{n} \sum_{t=1}^{n} Q_0$$

Where Z is the objective function, $Q_0(t)$ is the observed flow at time t, $Q_S(t)$ is the computed flow at time t, and Q_A is the average observed flow. The objective function is evaluated for all times t in the objective function time window.

4.7 San Tomas Creek Watershed

4.7.1 Continuous Model Results

Continuous discharge records from two gages on San Tomas Creek (above Williams Road) were used to calibrate the HEC-HMS model for the portion of the watershed studied. This portion of the watershed is just upstream of lower watershed that has concrete lined channels. Gage #24 is located on Upper San Tomas Creek just upstream of Williams Road; Gage #29 is located at Elwood Drive, see Figure 4-3. Due to the large quantity of data generated in the Upper San Tomas Creek models, this section will limit the discussion to results from these junctions.

In the lower portion of the watershed, the flow record for stream Gage 24 was used to calibrate the Pre-urban condition and the Existing condition models at junction J-6. Flow data was available from September 1955 to September 2004, and calibration was performed for the period from June 2002 through June 2003.

For the upper portion of the watershed, the flow record for stream Gage 29 was used to calibrate junction J3e of the Pre-urban condition model and junction J1 the Existing condition model. Flow data was available from October 1945 to October 1968 and calibration was performed for the period from October 1960 through October 1962.

5 Stability Assessment

This chapter summarizes the results of applying the stability assessment to the Thompson Creek, Ross Creek and San Tomas Creek sub-watersheds. These results from Ross and San Tomas Creeks, which are in the Central and Western portions of Santa Clara Basin, are compared to the Thompson Creek on the eastern portion of Santa Clara Basin.

The comparison answers the following question:

- Are the Thompson Creek, Ross Creek and San Tomas Creek results significantly different enough from each other to warrant a separate hydromodification standard, criteria and threshold, or can a single Basin wide standard be defined that applies to all watersheds in the Basin?

Sections 5.1 and 5.2 provide a summary of the background information related to the conditions and assumptions of the method. Section 5.3 describes the mechanics of the method. Section 5.4 describes how well the method works compared to the observed field conditions and presents the tools to be used for management decisions. Section 5.5 applies the method to future conditions and summarizes the results. Section 5.6 summarizes the findings of the Stability Assessment and Section 5.7 briefly states the conclusion on developing a Basin wide standard.

5.1 Overview

Increases in impervious surfaces caused by urbanization and the associated changes in runoff can increase erosion in streams. Development creates large percentages of impervious surfaces that are interconnected via rain gutters, storm drains, and open channels discharging into the creeks. The modification of stream flows due to increases in impervious surfaces and connectivity is considered the most significant cause of channel erosion in urban stream channels (Hammer, 1972; Booth et al. 1997; Doyle et al 2000; Bledsoe et al 2001). This modification to stream flows is termed as hydromodification. In addition to hydromodification, urbanization encroaches onto floodplains and channels, can change sediment sources and supplies, and alter vegetation patterns and densities that influence stream stability.

The stability assessment methodology is based on the premise that a balance among flow energy, sediment supply, and channel resilience must be maintained in order for the stream network to remain stable (Ontario, 2003). By applying this method and establishing management criteria, the intent is to maintain stream sediment transport and erosion processes, not to eliminate them.

Using continuous simulation, the method integrates the excess shear stress applied to the channel boundary over the total time (duration). Excess shear stress is defined as the amount of applied shear that exceeds the critical shear stress for initial motion of bed material or erosion of bank material. This integration is done over a rainfall record of 50-years in this analysis. The assessment method measures erosion potential by using an index representing

the effective work done by flow energy in excess of the amount required to transport the available sediment load.

To gain confidence that the method is a reliable predictor of stream channel erosion and instability, the method is used to compare model predictions to observed field conditions classifying the current eroded state of the stream channel (low, medium or high). An empirical relationship is derived that relates model predictions to the probability of having unstable channel conditions. Through this relationship, a threshold is defined that predicts the on-set of channel adjustment and erosion, which is then used to evaluate the effectiveness of management strategies. Chapter 5 discusses this relationship, compares and contrasts the results between the Ross Creek, San Tomas Creek and Thompson Creek subwatersheds.

Natural processes such as weathering, landslides, debris flows, and hill slope erosion supply sediment to the valley floor and stream channels. This material is transported downstream, broken down into smaller material, deposited, scoured, and continually reworked. The hydraulic energy of stream flow imposes a shear force that mobilizes the erodible soils and rock and helps shape the stream channel network. Over time, channels evolve to approximately stable equilibrium conditions that balance the imposed flow energy and sediment load with the channel boundary materials' ability to resist erosion. The processes of runoff and sediment transport interact with the boundary materials establishing cross-sectional geometry, longitudinal slope and planform. As vegetation co-evolves with the channel, it too influences channel stability and shape.

Minor episodic storms (including El Nino) small fires and landslides can also affect sediment characteristics of stream channels. Such minor episodic events occur at a frequency of one in 5 to 20 years in the Santa Clara Basin and can establish a new set of stream conditions and vegetation patterns that gradually return to the pre-event geometry over time. Major episodic events (large fires, major earthquake fault movement) may establish new set of stream conditions. Major episodic events do not occur every 100 years.

The stability assessment primarily addresses changes in watershed runoff patterns and does not specifically address changes in sediment or vegetation patterns. The methodology measures changes and impact from urbanization. It does not measure or recommend management measures for episodic effects associated with fires, seismic events or landslides. These influences would be accounted for under the geomorphic/ historic assessment, and characterization of the watershed and stream system.

5.2 Channel Stability and Equilibrium

5.2.1 Channel Stability and Thresholds

To test and verify the method, the stability assessment requires stable "*baseline*" conditions to compare to the existing conditions. Stable baseline conditions are being represented by pre-urban watershed conditions and modeling. Both the hydrologic modeling and the stream hydraulic modeling were completed using 1940s and early 1950s conditions on Ross and San Tomas Creek watersheds, and 1960s conditions on Thompson and Lower Silver Creek watersheds.

A stable channel is loosely defined as one that neither aggrades nor degrades, but instead maintains its average cross-section, planform, and profile features over time and within a range of variance. Several researchers have defined the equilibrium concept as one where the spectrum of discharges, slope, and channel geometry are adjusted to provide just the right energy to transport the sediment load supplied to the system (e.g., Knighton 1998).

Stream systems in equilibrium can tolerate a certain amount of variation in its flow and sediment loads through natural self-regulating mechanisms. A stable channel can tolerate short-term disturbances without significant change. A disturbance of sufficient magnitude and duration that exceeds the ability to self-regulate causes the channel to begin changes, which is defined as the "threshold of adjustment". Under such conditions, streams can migrate, widen or incise into underlying materials. Stream systems may never truly be in perfect equilibrium, but over the short time period of observation, stream systems tend to maintain consistent measurable characteristics, or if disturbed, streams tend to return to approximately their previous state.

The intent of the assessment method is to identify the threshold of adjustment where channels begin to incise or widen and the forces that cause this adjustment, accelerating the natural erosion and sediment transport processes. This threshold of adjustment is proposed as the measurable limit of allowable change in the watershed and used for management purposes.

5.2.2 Work Concepts

In addition to the equilibrium concept, there also is a range of flows that are considered most important in defining channel form, adjustment and controlling the rate at which sediment is transported through the stream system (Leopold et al. 1964). Leopold et al. (1964) showed that a large percentage of the "work done" is performed by frequent flow events of moderate magnitude defined as "geomorphically significant flows" in this report. Research has shown that urbanization significantly alters the frequency and duration of geomorphically significant flows (Hammer 1972, Hollis 1975). Bledsoe and Watson (2001) reported that the frequency of these flows increases by factors of 2.5 to 5 for watersheds with 18 percent impervious cover. MacRae et al. (1992, 1993) showed that the greatest increase in channel erosion results from increases in the small and moderate sized flow events – referred to as sub-bankfull flows. These small but frequent flow events have the energy to move sediment and erode stream bank material, and cumulatively have more influence over channel erosion and adjustment than infrequent larger storm events. Additionally, they can weaken the bank and therefore contribute to more extensive and severe slumping during larger storm events.

5.3 Methods

5.3.1 Effective Work Index

The stability assessment is based on measuring the magnitude of effective work index (W) by flows that exceed a specified critical value for the streambed or bank material. W is an index that represents the total work done on the channel boundary integrated over time. Note that the index includes a velocity term to accurately represent W in units of Work. The

Thompson Creek study was updated to include the index as shown below in Equation 1 and illustrated earlier in Figure 2-2.

This effective work index is defined as follows:

$$W = C \cdot \sum_{i=1}^{n} (\tau_i - \tau_c)^e \cdot V \cdot \Delta t \qquad (1)$$

where:

W = index of total effective work done over the length of flow record per square foot of bed or bank (ft-lbs/sq-ft).

C = a constant to convert equation to dimensional or dimensionless units of work, dependent on exponent e

n = number of flow records in a histogram of flows

τc = critical shear stress that initiates bed mobility or shear erosion (lbs/sq-ft).

τi = applied hydraulic shear stress, computed as ρgdS (lbs/sq-ft).

e = exponent that captures the exponential rise in stream power with flow (determined to be between 1 and 2.5, selected as 1.5 for watersheds in Santa Clara Basin)

V = mid-channel velocity (ft/sec)

Δt = duration of flow (in seconds) for each flow record

For τi ,
d = depth of water (ft), S = longitudinal slope (ft/ft),
g = gravity constant (ft/sec2) ρ = density of water (lb/ft3)

The time increment (Δt) is determined by generating a histogram of flows from the continuous hydrologic simulation results, which are hourly data. For each flow range (Bin), the histogram provides the count or duration of time that flows are within the designated flow range. For the average flow within a Bin, the depth, velocity, and shear stress are computed. Equation 1 is solved for each flow Bin, where the excess shear term is multiplied by the velocity (V) and duration (Δt) to compute the incremental effective work done by that specific range of flows. When equation 1 is summed over the 50-year flow record (histogram), the result (W) is a measure of the total effective work done on the stream channel boundary.

5.3.2 Determining Critical Values

Critical values of the streambed and stream bank provide a measure of the stream's resistance to erosion. Critical values for bed material reflect the onset of sediment transport. Critical

values for bank material reflect the onset of erosion of the bank, especially for weak stratigraphy layers. Streams (or boundary material) with larger critical values have more resilience to hydromodification. Chapter 3 described the physical properties of the observed streambed and bank materials, which are used to determine the critical values for the channel boundary. The bed was characterized by particle size distribution and depth. Stream banks were characterized by composition (percent clay, silt, sand and gravel).

For the different bed material sizes, critical values of shear stress and velocity for bed mobility were estimated using two methods: Shield's equation and from permissible velocity tables published in ASCE Manual No. 77 (1992). Table 5-1 lists the range and average of estimated critical velocities and shear stresses using each method for comparison. Ultimately, the average ASCE values were used in the stability analysis to maintain consistency between the values selected to represent the critical shear for bank material.

Table 5-1. Range of Critical Shear Stress and Velocity for Measured Bed Material in Thompson Creek

Study Area	D50 (mm)	Bed Slope	Critical Bed Shear Stress (a) (lbs/ft^2)	Critical Bed Shear Stress (b) (lbs/ft^2)	Critical Velocity (a) (ft/s)	Critical Velocity (b) (ft/s)
Thompson Creek Range	3.2 to 10	0.002 to 0.018	0.05 to 0.16	0.09 to 0.27	1.5 to 2.8	2.4 to 3.5
Average	6		0.09	0.14	2.1	2.8
Ross Creek Range	14 to 30	0.004 to 0.019	0.22 to 0.48	0.21 to 0.61	3.3 to 4.4	3.9 to 5.2
Average	20		0.29	0.35	3.9	4.3
San Tomas Creek Range						
Average						

a) Source: Computed using Shields Equation. Dimensionless parameter used = 0.047
b) Source: ASCE Manual No. 77, page 334, Figure 9.5.

The ability of a stream bank to resist erosion depends on soil materials, stratigraphy, vegetation density, root strength, the degree of cohesion, bank height, and slope. Boundary material also influences vegetation assemblages which in turn provide resistance to bank erosion. Stream channels bounded by clays, compacted silts, and loess are often more resistant to erosion and respond more slowly to hydrologic changes than channels bound by loosely consolidated sands and gravels.

The ASCE Manual No. 77 was used to estimate critical velocities and shear stress for the channel banks, including both the upper stiff clay layers and the lower weaker layers with sands and gravels. Table 5-2 lists the critical values selected for the different bank materials found in the Thompson Creek and Ross Creek subwatershed. The composition of the bank soils was estimated qualitatively in the field.

Table 5-2. Selected Critical Velocity and Shear Stress Values for Bank Material

Material Type	Critical Velocity (V_c ft/sec)	Critical Shear Stress (τ_c lbs/ft^2)
Riprap	8.0	1.6
Hardpan	6.0	0.67
Compacted Clays	5.0	0.5
Stiff Clays	4.0	0.32
Alluvial Silts	3.5	0.23
Firm Loam	3.5	0.23
Silty Loam	3.0	0.17
Sandy Loam	2.5	0.12

5.3.3 Hydraulic Computations

Hydraulic calculations convert the flow rates estimated by the hydrology model to flow depth, velocity, and shear stress based on cross-section geometry and slope. Channel hydraulics are computed using normal flow assumptions. Each cross-section is treated independently from the others; thus backwater effects are not considered. The computations are completed following the Army Corps of Engineers HEC-2 method, where conveyance (K) is computed and summed between individual survey points. A HEC-2 (HEC-RAS) model was not developed. The depth, velocity, and shear stress used in the stability assessment are taken from the main channel not including overbanks or floodplains. The central channel velocity and shear stress more correctly represent the actual values imposed on the channel bed and toe of banks where shear erosion is occurring.

The following equations are used for the hydraulic analysis:

$$Q = 1.49 \cdot K \cdot \sqrt{S}$$

$$K = \sum \frac{AR^{2/3}}{n} \qquad R = \frac{A}{P}$$

where

K = Conveyance

R = Hydraulic radius

P = Wetted perimeter

Conveyance is computed for each element of the flow area defined between two cross section survey points. The total conveyance (K) is then determined by summing the conveyance between individual elements. Once K is known, the discharge is computed. The

computation begins by computing the flow area (A) and wetted perimeter (P) for a know depth of water, followed by computing *K* and then discharge (Q).

For the stability assessment, hydraulic tables were generated for each cross section by incrementing stage in 6-inch increments up to the maximum surveyed elevation. This stage-discharge table includes hydraulic radius, flow velocity, bed shear stress, flow area, and a composite roughness coefficient. The stage-discharge table was used in conjunction with the histograms to predict the hydraulic variables (by interpolation) given the model flows.

Selecting Roughness Coefficients

Roughness coefficients, required for hydraulic calculations, were first estimated in the field and then adjusted in the office using photographs taken at each individual cross-section. Coefficients were estimated using Cowan's method as described in Chow (1959) and others. Cowan's method sums individual roughness elements of the stream boundary. For example, bed material and form, irregularities in the banks, variations in cross-section, obstructions, and vegetation density are elements considered to derive the overall roughness coefficient. For the hydraulic computations, coefficients were selected to represent low flows in the mid-channel, bankfull flows, and flood flows, and vary by elevation and location.

5.3.4 Erosion Potential

The effective work index for stable stream channels under pre-urban conditions (or drainage areas as close to undeveloped as possible) is compared to unstable channels under urbanized conditions. The comparison, expressed as a ratio, is defined as the Erosion Potential (Ep) (McRae, 1992, 1996) and illustrated earlier in Figure 2-3.

$$Ep = \frac{W_{unstable}}{W_{stable}} \qquad (2)$$

$W_{unstable}$ = work index for a stream section determined to be unstable

W_{stable} = work index for a stream section determined to be stable

The Ep is used to develop the empirical relationship between Ep and the observed stability of stream channels.

Predicting the erosion potential for future development projects and for management purposes, the Ep ratio would be written as:

$$Ep = \frac{W_{post}}{W_{pre}} \qquad (3)$$

W_{post} = work index estimated for proposed development

W_{pre} = work index measured for the pre-development condition

The concept here is that during implementation of the HMP, the baseline pre- condition is the *Existing* condition. The existing condition may, or may not, be stable. When existing conditions are not stable because of current development, implementation of the HMP requires that Ep not increase and make the observed erosion worse.

5.3.5 The Erosion Potential Chart

The measured Work Index and Ep values for pre-urban and existing land use conditions are compared to the field classified erosion condition (stable/low and medium/high). The numeric value of each parameter is plotted on a vertical chart by group and is referred to as the Erosion Potential Chart. Groups with indistinguishable measures are combined; groups that are distinguishable suggest potential thresholds for management purposes. Earlier results from Thompson Creek showed the above groupings to be appropriate.

As with any environmental data, there is always variation in results and some level of uncertainty in mathematical expressions that attempt to predict environmental conditions. Predicting the on-set of erosion and channel instability is a complex matter. Overlap in groupings illustrates the uncertainty in the effective work index and Ep. The Erosion Potential Chart will be described in more detail under Results.

5.3.6 Logistic Regression

It is not possible to use a linear regression technique to accurately predict the risk of erosion when there are only two discrete states that distinguish between stable and unstable conditions. Logistic Regression is a better methodology which is used to predict the probability, or likelihood, of having unstable channel in a two discrete state model, based on a measure of Ep. This technique allows us to establish thresholds based on the likelihood of having unstable channel conditions based on a probability of having unstable conditions. The Logistic Regression technique will be described in more detail under Results. Further discussion is included in Appendix A.

5.4 Results

The results are presented for the Thompson Creek Watershed, and then for the Ross Creek and San Tomas Creek watersheds. The Ross Creek and San Tomas Creek results are then discussed in comparison to the Thompson Creek. Sub-section 5.4.1 provides a discussion on percentage of watershed imperviousness specifically as it relates to Segment 5 in Thompson Creek and for East Ross Creek where field observations noted a transition from stable channel conditions to unstable conditions. The literature frequently reports observed channel and riparian impacts once the percent of imperviousness reaches a certain level (frequently stated as 10%). Sub-section 5.4.1 tests these conclusions in the Santa Clara Basin. Sub-section 5.4.2 summarizes the result of the effective work computations and compares the computed index to the observed field classification for erosion. Close agreement would indicate that the effective work index can predict the likelihood that a stream channel would be stable or unstable. Sub-section 5.4.3 discusses results for the Erosion Potential (Ep) and sub-section 5.4.4 discusses the logistic regression results, which are used to derive the threshold of adjustment.

5.4.1 Percentage of Watershed Imperviousness

Booth et al. (1990, 1997) reported finding a good correlation between the onset of channel instability and recent increases in percent imperviousness. According to Booth, western Washington lowland streams display the onset of degradation at a consistent level of development, and instability is observed when the effective impervious area increases to 10 percent or more.

The project team compared the percentage of impervious area for cross-sections in Thompson Creek and Ross Creek. Figure 5-1T and 5-1R presents an illustration showing the cross section location, contributing sub-catchments, and percent impervious area, for Thompson Creek and Ross Creek, respectively.

In Thompson Creek, the project team compared the percentage of impervious area for cross-sections TC5-4, TC5-5, TC5-6 and TC5-7 in Segment TC5 under existing conditions. In Ross Creek, the project team compared cross sections in East Ross (ER-1, -2, and -3) as well as one on the main stem of Ross Creek (RC-8).

The percent of impervious area associated with cross-sections in the upper parts of the watersheds, where the cross-sections transition from stable to unstable conditions is shown in Table 5-3 and in Figures 5-1T and 5-1R.

Table 5-3. Summary of Percent Impervious Area in Transition Segments of Thompson Creek and Ross Creek subwatersheds

Ross Creek	Current % Impervious	Erosion Rate Classification	Thompson Creek	Current % Impervious	Erosion Rate Classification
ER-3	1.2	Stable	TC 5-7	1.6	Stable
ER-2	5.8	Medium/High	TC 5-6	6.9	Low
ER-1	8.9	High	TC 5-5	9.4	High
RC-8	18.7	High	TC 5-4	11.1	High

The percent of impervious area associated with cross-sections TC5-4, TC5-5, T5-6, and T5-7 are 11%, 9%, 7% and 1.6%, respectively. The field study classified erosion along cross-sections TC5-4, TC5-5, TC5-6, and TC5-7 as "high," "high," "low", and "stable", respectively. Furthermore, cross-section TC5-5 is just beginning to show the signs of instability, where the bed has recently incised six-inches, reach wide.

The percent of impervious area associated with cross-sections RC-8, ER-1, ER-2, and ER-3 are 19%, 9%, 6% and 1.2%, respectively. The field study classified erosion along these sections as "high", "high," "medium/high", and "stable", respectively. According to the field crews, cross section ER-2 is mostly medium.

Given the results for both Ross Creek and Thompson Creek, it appears that a similar relationship between percent imperviousness and stream channel instability might exist in the Santa Clara Basin as that found for Western Washington by Booth. The transition between stable and unstable conditions occurs between 5 and 10 percent impervious.

5.4.2 Testing the Assessment Method's Ability to Predict Observed Instability

This section first discusses the effective work curves, which suggests which flows are responsible for doing the most work on the channel boundary, and when compared between pre-urban and existing, illustrate how urbanization changes these important flows.

Effective Work Curves

Thompson Creek

Figure 5-2T present effective work curves and cumulative effective work curves for the pre-urban watershed conditions for a sub-set of cross sections in Thompson Creek. These results represent the stable baseline condition upon which the existing and future conditions are compared and used to evaluate potential instabilities. Figure 5-3T presents the computed effective work curves and cumulative effective work curves for cross-sections in Segment 5 under existing conditions.

These curves illustrate which flows are doing the greatest amount of work on the stream channel and are responsible for the greatest amount of channel incision and toe erosion. For the urbanized cross section TC5-1, -2, -3, -4, and -5, flows between about 15 cfs and 80 cfs are responsible for greatest percentage of the erosion and instabilities in this segment. This is observed by the larger magnitude of the effective work index between 15 cfs and 80 cfs. For cross sections TC5-6 and TC5-7, with little development, there is much less influence of small to moderate sized storm on the overall effective work. The important geomorphically significant flows begin at about 15 cfs but extend to 250 cfs and 190 cfs for TC5-6 and TC5-7, respectively. These curves illustrate the effect urbanization has on increasing the frequency and duration of small to moderate sized storms and its affect on *Work*, and is consistent with results presented by others in the literature (Hammer 1972, Hollis 1975, Bledsoe and Watson 2001, MacRae et al. 1992, 1993).

Ross Creek

Figure 5-2A presents effective work curves for select cross sections for the pre-urban, existing, and future watershed condition for Ross Creek. The pre-urban results (late 1940's and early 1950's), when the watershed was agricultural and undeveloped, represent the stable baseline condition upon which the existing and future conditions are compared and used to evaluate potential instabilities. These curves illustrate which flows are doing the most work on the stream channel boundary. As observed with Thompson Creek, the small to moderate magnitude flows have the greatest influence on channel shape and erosion potential. For example, under pre-urban condition for Ross Creek, flow from 25 cfs to about 240 cfs accounts for 90% of the total work done. This range of frequent flow events constitutes the geomorphically significant flows for Ross Creek.

The change between the existing curves and the pre-urban curves illustrate which urban flows are doing the greatest amount of work on the stream channel and are responsible for the greatest amount of channel erosion. The work done at cross section RC-8 for example increases the most dramatically when compared to the other cross sections. The maximum work done under pre-urban conditions is about 200,000 ft-lbs/sq-ft at 40 cfs, which increases to 640,000 ft-lbs/ sq-ft under existing conditions.

Figure 5-2B present the cumulative work curves for Ross Creek, sections RC-3 to RC-8. These curves illustrate which flows are contributing the most to the overall total work done on the stream channel. The figure also shows the magnitude of the 2-year, 5-year, and 10-year peak flows. This figure illustrates how much work is done for flows of different magnitudes. For example, 70% to 80% of the work is done by all flows up to the 2-year peak flow. Between 92% and 94% of the work is done by flows up to the 10-year peak flow. Provision C.3.f.iv of the permit requires the HMP to specify the range of storms for which the HMP applies. From Figure 5-2B, the Project Team is recommending that flows be managed from zero up to the pre-project 10-year peak flow.

Figure 5-3 presents effective work curves for the pre-urban, existing, and future watershed condition for East Ross Creek. These curves show similar trends as well as some differences. ER-1 and ER-2 both show increases in work in the low to moderate flow range. However, ER-3 actually shows decreases in work and in the range of flows. It may be that the construction of impervious surface upstream from ER-3 creates the bimodal hydrograph phenomenon where runoff from the impervious surfaces travels faster and exits the catchment before the remaining forested catchment area is able to contribute to flows.

Figure 5-4 presents effective work curves for the pre-urban, existing, and future watershed condition for RC-1 and RC-2. These two cross sections are located in a portion of the channel that was excavated where no stream channel existed naturally to route flows to Guadalupe River. These two cross sections illustrate results for an earthen engineered channel with an erodible boundary. The maximum work for the existing conditions is not too much different than the maximum work under the pre-urban conditions. The existing curves are a little broader than the pre-urban curves. These cross sections show signs of significant past erosion. At present, they have low to medium erosion presently. Figure 5-4b illustrates what the work curve would look like using existing flows and the design cross section. This curve suggests that much more work was done on the channel in the past than what is predicted today. This is consistent with the field observations that these cross section show signs of past erosion with about 2-feet of incision and bank erosion. Today, these cross sections are wider and deeper and may be reestablishing a state of equilibrium.

Like Thompson Creek, the work index computed in Ross Creek increases more so for smaller flows than larger ones. However, the effective work in Ross Creek does not increase as dramatically in the low flow range as predicted for Thompson Creek. Effective work in Ross Creek exhibits a more or less uniform increase (percentage wise) within the low to moderate flow range (roughly 25 cfs to 500 cfs). These results may reflect the hydro-geomorphic nature of Ross Creek. Ross Creek is mostly perennial, larger lot sizes per dwelling, and larger bed material than Thompson Creek. The upper watershed is forested

and has lower perviousness, unlike Thompson Creek whose watershed is mostly grass lands and more pervious.

5.4.3 Evaluation of the Field Designated Erosion Classification

Figure 5-5 plots the total effective work index with erosion classification for Thompson Creek. Estimates for pre-urban conditions are also shown for comparison. The total effective work index is computed by the model and the erosion classification is determined in the field for existing conditions. The distribution of stable/low versus unstable – medium/high along the total effective work index suggests a strong correlation between the computed work index and the field designated erosion classification.

Figure 5-6 compares the total effective work index (W) with the erosion classification for existing conditions based on field observation in Ross Creek. Unfortunately, all but two cross sections were classified as medium/high, which makes it difficult to make strong statements regarding correlation to field observations. Considering the two cross sections in the stable/low classification, RC-7 is just upstream of a grade control structure installed in 1958, and ER-3 is predicted to have a decrease in work due to shifts in the flow histograms.

The range in magnitude of the effective work index for Thompson Creek is greater than what is computed for Ross Creek. To understand this better, it is necessary to look at a number of factors:

1. Critical shear stress in the two watersheds: For the same total shear stress applied, the critical shear stress at Ross Creek is higher, thereby resulting in a lower value of total effective work. The critical shear stress on Ross Creek is higher because of larger D50 of the bed material. The D50 is larger because of differences in the geomorphology, the mountain ranges and perennial nature of flow in Ross Creek, which does not allow as much of the fines to settle out. Tables T3-3 and R3-3 lists the D50 values for Thompson and Ross creeks

2. Age of development and channel response: Another possible explanation is the difference in the time period of adjustment since intense development has taken place. Possible explanations for these differences include cross section differences (channel geometry and slope have had more time to adjust), watershed size and flow magnitudes (Thompson Creek has twice the watershed area and flows), and stream geomorphology differences (Ross Creek has larger bed material). Considering sections RC-1 and RC-2, there is evidence that the erosiveness of flows through this reach has decreased over the years as the channel expanded. Ross Creek has had roughly 20-years longer to adjust to development (from the 1960's & 1970's) than Thompson Creek (high rate of development continuing from 1980's to present). Further analysis of more cross sections would likely show the same pattern.

5.4.4 Erosion Potential (Ep)

Figure 5-7 presents the Erosion Potential Chart for Thompson Creek. As with Figure 5-5, there is a strong correlation between computed Ep and the field-designated erosion

classification. Those sections classified as having medium/high observed erosion characteristics plot on the chart above those classified as stable/low with only two outliers.

Figure 5-8 presents the Erosion Potential Chart for Ross Creek. As with Figure 5-6, there is a suggested correlation between the predicted erosion potential and the field-designated erosion classification. Those sections classified as having medium/high observed erosion characteristics plot on the chart above those classified as stable/low (even with only two points). The minimum value of the medium/high group is 1.6. RC-7 is just upstream from a grade control structure installed in 1958 as part of the channel improvement projects. ER-3 is reduced as described earlier.

Figure ST5-8 the Erosion Potential Chart for San Tomas Creek, which plots very similar to Ross Creek. San Tomas has four cross sections that are classified in the stable/low category and five that are classified as medium/high. San Tomas, however, includes a medium/high section with an Ep ratio of 1.1. Four cross sections in San Tomas (ST-7, 8, 9, and 10) were not used because they were located near an active fault line and the area was determined to be naturally highly erosive. ST-7 is located immediately downstream from an outfall, which is causing localized scour, so it too was not included in the final pooled analysis.

By comparing the results for Thompson Creek, Ross Creek and San Tomas Creek, the Ep values at which cross-sections transition from stable to unstable rates of erosion are similar. This should allow for pooling the data for logistic regression analysis on watersheds in different parts of the Santa Clara Basin.

5.4.5 Selecting a Threshold for Management

One of the project objectives is to determine a *threshold* value of Ep and to recommend management strategies. Earthen engineered channels can potentially be designed with the same threshold applied. This section describes how a value for the threshold can be determined based on the likelihood that the stream channel becoming unstable. Results are discussed for the pooled San Tomas Creek, Ross Creek, East Ross Creek, Yerba Buena and Thompson Creek data.

Figure 5-9 presents the probability curve of the erosion potential, based on a logistic regression analysis for the pooled data from the Ross Creek subwatershed combined with the results from San Tomas Creek and Thompson Creek subwatershed. This curve can be used to determine the probability of having unstable conditions given a value for Ep. If for example the Ep was computed to be 1.7, the chance of having unstable channel conditions is 50:50. Channel sections with Ep values in excess of 2 have a 77 percent chance of becoming unstable.

For evaluating management strategies, it is recommended that the probability (risk) of erosion be examined for an Ep range of 0.8 to 1.2. These values are presented below in Table 5-4.

One way to think about the risk of instability is to think about it in terms of the number of streams that could still become unstable. For example, a 20% risk implies that 1 in 5 streams will become unstable and lead to erosion problems. A 10% risk implies that 1 in 10 streams

could still become unstable. Given these results, it is recommended that HMP management strategies and policies be focused on maintaining a level of risk at 10% or less.

Table 5-4. Summary of Corresponding Ep values and Estimated Risk of Channel Instabilities

Ep	Risk of Channel Instabilities
0.8	5%
0.9	6%
1	8%
1.1	10%
1.2	16%

It is recommended that the District not allow a stream condition in eroding segments to worsen (increase the Ep value further). For this, it is necessary for future developments to match the entire flow duration curve to pre-project conditions and match the work done on the channel. The value of Ep when such matching is done is Ep = 1.0.

It should be noted that the Ep values computed above are based on the sum total of changes that take place in a watershed. This includes the impact of roads, compaction of land, and other development not subject to HMP requirements. For a BMP that is limited to the boundaries of the development, the impacts from areas not subject to HMP requirements have to be mitigated by additional measures such as erosion in-stream modifications, or selecting an Ep less than 1.

An Ep of 1.0 still leaves a residual risk of channel erosion from uncertainties in the data and design (Figure 5-9). This risk has to be borne by the agency maintaining the stream channel downstream. Stream erosion protection measures can be installed to manage the risk.

A discharger may also select an Ep larger than 1. This would increase the risk of erosion downstream which will be have to be managed through additional erosion protection measures. For example, selecting an Ep of 1.2 results in a risk of erosion of 14 percent. The discharger could be assessed the costs for managing this risk. Alternatively, a discharger may select an Ep less than 1. This strategy incorporates a safety factor and minimizes the risk downstream. The discharger could be compensated through return of funds to be contributed towards hydromodification as part of a development.

5.4.6 Analysis of Critical Flow (Qc)

This section describes the determination of the Critical Flow (Qc) for Thompson Creek and Ross Creek. Critical flow is defined as the stream flow (cfs) that produces the critical shear

stress (τc) and initiates bed movement. Table 5-5 lists the estimated critical flow at cross sections near locations where flood frequency results were generated.

Critical flows for Thompson Creek range from 3 cfs to 40 cfs depending on the location listed. The average critical shear of 0.14 lbs/sq-ft is used for most all sections. According to the sediment sampling data collected in Thompson Creek, estimated critical flows is estimated to range from 1 cfs to 18 cfs. Critical flows for Ross Creek range from 15 cfs to 25 cfs depending on the location listed. In Ross Creek, critical shear values ranging from 0.23 to 0.38 lbs/sq-ft depending on location were used in the analysis.

In order for the critical flow to be useful to dischargers, the critical flow in the stream must be partitioned or related to an on-site project based variable. For this analysis the in-stream critical flow was related to the pre-urban 2-year peak flow (Table 5-5). Because computations involving sediment data, critical shear values, and roughness coefficients are highly variable and subjective, the critical flow was generalized as being 10% of the 2-year peak flow.

Table 5-5. Summary of 2-year Peak Flows and Estimated Critical Shear Stress Values

Subwatershed and Location	Pre-Urban 2-Year Peak Flow (cfs)	Cross Section	Critical Flow (Qc) (cfs)	Percent of 2-Year Peak
Thompson Creek				
J-1	56 (a)	TC5-7	10	18
J-5	189 (100)	TC5-1, 5-2 & 5-3	3, 5, 7	2, 3, 4
J-12	530 (500)	TC1-1, 1-2	40, 40	8
Yerba Buena	69	YB-6, YB-7	5, 4	7, 6
	57	YB-2, YB-4	3, 10	5, 18
Ross Creek				
J-1	209 (b)	RC-1, RC-2	25, 25	12
J-5	139	RC-6, RC-8	20, 15	14, 11

a) Thompson Creek 2-year peak flows were generated from synthetic design storm hydrographs as described in Chapter 4. Flows in parenthesis are estimated from the 50-year continuous model records where flood frequency data were generated.

b) Ross Creek 2-year peak flows were generated from the 50-year continuous model records. Design storm hydrographs were not generated for Ross Creek.

5.5 Predicting Erosion Potential under Future Build-Out Conditions

5.5.1 Thompson Creek

Table 5-6 presents the results for total effective work index and erosion potential under future conditions (build-out per the current San Jose General Plan 2020) for select stream sections. The percent increase in the work index from existing conditions is also presented. These results suggest that future development will increase the current erosion potential by an average of 26 percent.

The important change to note is that Ep for cross-sections TC5-6 and TC5-7 shifted from about 1 to an Ep of 1.3 and 1.2, respectively. Future development increases the probability that cross-sections TC5-6 and TC5-7 will become unstable. If this reach of Thompson Creek were to become unstable, then erosion of the bed and bank would contribute sediment to downstream reaches.

Table 5-6: Percent Increase in the Erosion Potential under Future Build-Out Conditions

Cross Section	Ep		
	Existing	Future	% Increase
TC1-6	5.4	6.7	24%
TC2-1	2.4	3.3	37%
TC2-3	3.7	4.8	29%
TC2-6	9.7	12.2	26%
TC2-9	4.1	5.3	31%
TC2-10	2.0	2.6	30%
TC3-5	10.4	11.6	12%
TC3-7	2.8	4.0	45%
TC5-2	4.0	5.4	35%
TC5-3	4.6	6.0	33%
TC5-4	8.2	10.5	30%
TC5-5	5.2	6.8	31%
TC5-6	0.9	1.2	10 %
TC5-7	0.9	1.3	12 %

5.5.2 Ross Creek

In this section, the results for the future build-out condition of Ross Creek and East Ross Creek in the Ross Creek sub-watershed are presented and discussed as an example of changes that can be experienced with future development. Sub-section 5.5.2.1 summarizes the result of the effective work computations and compares the computed index to the observed field classification for erosion. Sub-section 5.2.2.2 discusses results for the future condition Erosion Potential (Ep).

5.5.2.1 Effective Work

Figure 5-2, presented earlier, shows the effective work curves for the pre-urban, existing, and future watershed condition for Ross Creek. The change between the existing curves and the future curves illustrate which additional urban flows will do the greatest amount of work on the stream channel and be most likely responsible for the increase in erosion in the future build-out condition. The work done at cross section RC-8 for example increases from pre-urban to existing to future conditions, when compared to the other cross sections. The maximum work done under future conditions is 920000 ft-lbs/ sq-ft, a 44% increase from existing conditions.

Figure 5-3, as presented earlier, shows the effective work curves for the pre-urban, existing, and future watershed conditions for East Ross Creek. These curves show similar trends as for Ross Creek. ER-1, ER-2, and ER-3 all show increases in the low to moderate flow range from existing to future condition due to the projected increase in impervious surface area within East Ross Creek.

Figure 5-4, as presented earlier, shows the effective work curves for the pre-urban, existing, and future watershed conditions for RC-1 and RC-2. While these cross sections are wider and deeper today, and may be reestablishing a state of equilibrium, the future condition curves do predict an increase in the effective work on these engineered channels which could correspond to additional erosion as the channel continues to adjust to a new state of equilibrium.

5.6 Findings

5.6.1 Effective Work

- The predicted effective work index (W) is capable of distinguishing between stable stream sections (pre-urban, stable/low existing conditions), and unstable stream sections (medium/high existing conditions).
- In all watersheds studied, the more frequent small to moderate magnitude flows have the greatest influence on channel shape and erosion potential.
- The critical discharge (Qc) upon which bed material begins to move in quantity is about 10 to 15 cfs for Thompson Creek and 20 to 25 cfs for Ross Creek. In both instances, it is 10 percent of the peak 2 year flow.

5.6.2 Erosion Potential (Ep)

- The Ep methodology is capable of distinguishing stable/low eroding conditions from medium/high eroding conditions for San Tomas Creek, Ross Creek and Thompson Creek subwatersheds (including East Ross Creek and Yerba Buena Creek).
- The Ep ratio between the subwatersheds is similar such that the Ep data for Thompson Creek, San Tomas Creek and Ross Creek can be pooled together to evaluate the threshold of adjustment.

5.6.3 Threshold of Adjustment

- Logistic Regression provides a statistical tool to relate the likelihood of stream channel instabilities to the computed Ep ratio.
- Because of the variability and uncertainties in the analysis and results, an Ep=1 is estimated to have a 8 percent chance of still being unstable. An Ep = 1.2 has a 15 percent chance of erosion.

- One way to think about the level of acceptable risk is to consider how many streams will become unstable given the level of risk. For example, given a risk of 20% means that 1 in 5 streams will still become unstable. Given a risk of 10% means 1 in 10 streams will still become unstable.

- Risks that are not captured in on-site or off-site designs have to be managed through in-stream erosion protection measures downstream.

- Since the value of Ep in the channel is based on all the changes in the watershed, including those outside the development, consideration has to be given to changes in runoff from compaction and impervious area addition outside the development subject to HMP requirements (such as service roads to the development).

5.7 Implications for a Watershed Wide Standard

The Ep ratio for watersheds studied are not significantly different to warrant separate hydromodification standard, criteria and threshold. Differences in the physical conditions between watersheds are accounted for in the assessment methodology.

6 Application of Methodology to all Watersheds

6.1 Summary

Provision C.3.f. of requires Co-permittees to develop a Basin wide HMP, and a single standard is preferred for uniformity and ease of implementation throughout the Basin. However, geomorphic theory suggests that watersheds and streams with different climate, physiography, soils and vegetation, could have different thresholds of adjustment and thus require different standards. As a result, the Basin was sub-divided into three regions with similar characteristics: the Coyote Creek watershed, the Guadalupe River watershed, and western watersheds. The Thompson Creek and the Ross Creek subwatersheds cover the Coyote Creek, Guadalupe River regions. San Tomas will cover the western watersheds.

Among the many questions being addressed in the development of the HMP, are questions addressing the need for multiple standards and the implementation of the HMP Basin wide. One such question is:

- Are the Ross Creek, San Tomas Creek and Thompson Creek systems significantly different enough from each other to warrant separate hydromodification standards, criteria and thresholds? or

This chapter summarizes the factors used to determine that a Basin wide standard is appropriate. It discusses what the range of storms should be for a basin wide standard. Also, based on the residual risk of erosion following impervious area addition presented in Chapter 5, it discusses the need for limited in-stream erosion protection measures downstream of HMP basins which are designed with Ep > 0.8. These have to be installed by the District to insure that the stream is not subjected to higher risk of erosion after development.

6.2 Justification for a Basin Wide Standard

One question to be addressed is "why do the results for Ross Creek seem to be consistent with the results for Thompson Creek?" Especially since these two subwatersheds have different climatic, physiography, soils and vegetation characteristics; and geomorphic theory suggest these stream systems could have different management strategies. A related question was raised during the preparation of the Literature Review - that is the idea that the Santa Clara Basin conditions are different from other regions that hydromodification assessment methods developed in other regions would not be applicable to Santa Clara. A member of the Peer Review Team[13] commented that this is a reasonable question to ask but one that can be resolved. Since models are calibratable and if the relevant processes are described adequately, methods from other regions in the Country can be used. The important point is that if the hydromodification assessment methodology incorporates the relevant physical

[13] The Peer Review Team includes Professor Matt Kondolf, UC Berkeley; Professor Tom Dunne, UC Santa Barbara, and Professor Brian Bledsoe, Colorado State University. The Peer Review Team is part of the SCVURPPP HMP project.

processes, than differences between regions - or differences between subwatersheds - are accounted for and a model can produce consistent results.

Another way this might be described is that the probability curve uniformly expresses the likelihood that Santa Clara Basin streams will become unstable given an increase in work done from the pre-urban condition. This is a significant conclusion that supports the development of a single Basin wide standard and management strategies based on maintaining the in-stream effective work.

6.2.1 Factors Used to Justify a Basin Wide Standard

The factors that were considered include the estimated erosion potential values between subwatersheds and the range of flows to be managed for hydromodification.

Erosion Potential

Chapter 5 discussed the estimated erosion potential for each of the test subwatersheds. One important element of these estimates is that the range in values does not differ significantly between subwatersheds. Even though the subwatersheds have different climatic, physiography, soils and vegetation characteristics; the erosion potential (Ep) is normalized on each subwatersheds own baseline conditions. Thus difference in critical shear stress between stream systems for example is accounted for in the baseline condition. Different stream systems can have different absolute values of work (higher or lower), but have similar tolerances for change. Ross Creek may be more resilient than Thompson Creek but responds to hydromodification at about the same percentage increase from its pre-urban condition (refer to Section 5.4, Chapter 5).

Critical Flow

Chapter 5 discussed the determination of the Critical Flow (Qc) for Thompson Creek and Ross Creek. Critical flow is defined as the stream flow (cfs) that produces the critical shear stress (τc) and initiates bed movement. The critical flow in the stream must be partitioned or related to an on-site project based variable. As described in Chapter 5, the in-stream critical flow was related to the pre-urban 2-year peak flow (Table 5-5) and was generalized as being 10% of the 2-year peak flow.

The results for Thompson Creek and Ross Creek are similar so that separate definitions for Qc is not required (refer to Section 5.4, Chapter 5).

Range of Flows for Management Purposes

The range of flows for management purposes has been defined as ranging from Qc up to the pre-urban 10-year peak flow. Qc is discussed in the previous paragraph. This section discusses the upper limit. According to Provision C.3.f.iv requires the range of storms (or flows) for which the HMP applies. Chapter 8 discusses the derivation for the upper limit.

The upper limit on the range of storms has been determined by evaluating the percent that different flow magnitudes contribute to the total work done on the channel boundary. The

low flows contribute the most work, whereas high flows contribute less. As discussed in Chapter 5, approximately 90% of the total work on the channel boundary is done by flows between Qc and the pre-urban 10-year peak flow magnitude. Flows greater than the 10-year rate contribute only 10% of the total work.

6.3 Conclusions

The results summarized in Chapter 5 found that Ross Creek (and San Tomas Creek) is not significantly different from Thompson Creek and does not warrant a separate hydromodification standard, criteria or threshold of adjustment.

Although the absolute magnitude of the computed work indices is different, the normalized Ep ratio is consistent between subwatersheds. The transition between stable and unstable channels occur between and Ep of 1 and 2.

The estimates for the *Range of Storms* to manage (Qc to the 10-year peak flow) is consistent between subwatersheds. The value of Qc is dependent on the bed material characteristics of each subwatershed. However, specifying Qc for development as 10% of the 2-year peak flow is consistent between subwatersheds.

7 Exempt Areas Delineation

7.1 Background

Provision C.3.f.ii allows for exempting discharges from new and significant re-developments into creeks or storm drains where the potential for erosion, or other cumulative impacts, in highly developed watersheds is minimal. The Project Team staff evaluated several methods and determined that a combination of two methods described below take into account the cumulative effects of upstream development at any point along the stream system. They can be used to determine non-exempt portions of the watershed, based on current measurements of unstable conditions as a result of erosion, and the magnitude of development above current levels that will destabilize additional segments of unhardened channels downstream. By delineating these areas in each watershed where there is or there can be significant impact from future development, the rest of each watershed can be delineated as exempt.

7.2 Methods

The Project Team has developed and tested two separate, but supporting assessment methodologies to delineate exempt areas within the Santa Clara Basin. These two methods are:

1) Determination of the most downstream limit (MDL) of erosion in each watershed, upstream of which hydromodification controls are required;

2) The increase in percent imperviousness in the future above existing levels at which the currently stable segments downstream of MDL may destabilize in the future.

It should be clear that neither method relies on any assumed value of future imperviousness. Both methods are based on actual field measurements of current erosion rates and impervious area (through infrared imaging). The conclusions are not dependent on any assumptions regarding the magnitude of future build-out.

The creeks in Santa Clara Basin were evaluated to identify and characterize the downstream-most extent (segment) of reach-wide erosion[14] in each watershed, where the rate of erosion is above what is considered 'typical' and 'stable', and is causing instability of the channel bed or banks of the creek. This segment has been termed as the most downstream limit (MDL) of erosion of the creek. The MDL defines the point where current active erosion is observed and where any further increase in watershed development will exacerbate the observed erosion problems. Developments that discharge stormwater above this point cannot be exempted from the HMP unless other measures are taken to mitigate the expected impacts.

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[14] Reach-wide erosion can be recognized by similar types and magnitudes of bed or bank distress (such as incision, outside-bend erosion, widening, etc.. In contrast, point- or local erosion is related primarily to factors influencing one particular location, such as a logjam or a bank weakness specific to that individual location.

The concept of the MDL is to locate a point in the stream system where it is definitive that the risk of channel erosion increases as a result of future development. Downstream from the MDL and in stable segments between MDLs, it becomes less certain that future development will, or will not, increase the risk of erosion. Full scale Basin wide hydrologic and hydraulic modeling would allow the computation of Ep within every stream and river segment, however, this is costly and time consuming and a simpler method is needed. Therefore, an assessment of the change in impervious surface area between existing and future development conditions is recommended. The concept of this approach is to identify a point in the stream system where the increase in percent impervious surface area as a result of future development is "minimal". Minimal impact is assumed to occur when the projected future build-out is small relative to current development conditions. This approach requires a criterion, or limit on the increase in percent imperviousness, to be used to determine exempt and non-exempt areas. This criterion is derived from the work analysis in Thompson Creek and from review of the literature.

In the absence of shear stress analysis, the Project Team recommends that a 3% change from existing to future conditions be set as the limit until shear stress analysis can be conducted. Justification of this value is described in this chapter.

A map of the Basin showing the locations of the MDL and the watershed upstream of MDL is included (Figure 7-1). The next section describes these two complimentary methods in greater detail.

7.2.1 Literature Review

Using continuous hydrologic modeling, King County studied the affects of development on higher order streams in the Cedar River Basin (1993) and Bear Creek Basin (1989) in Western Washington. The King County results suggested that flow control ponds would not have much effect on the duration of flows exceeding their Qc value for 4th order streams and larger. These studies however were not focused on exempt areas, were not conclusive and had a number of qualifications, which lead to the recently published report conducted by Herrera Environmental Consultants (HEC, April 2004).

In 2003, Inter-Fluve published a study on the basis for requiring flow control to protect Eastern Washington streams. Inter-Fluve proposed that discharges to 5th order streams and larger could be exempted from HMP controls. Inter-Fluve cited two lines of reasoning in developing their recommendation: 1) research in Maryland suggesting that 4th order streams and larger are not significantly affected by urbanization, and 2) research in Washington that suggest channel instabilities begin when the watershed imperviousness reaches and exceeds 10%. Inter-Flue eventually recommended that discharges to streams (rivers) of 5th order with less than 5% total impervious area could be exempted. Both criteria must be met for a project to be exempted.

The Washington State Department of Transportation, in cooperation with the Washington Department of Ecology, conducted a study of exempt area delineations in Western Washington (HEC, April 2004). HEC also reviewed the available literature and concluded that the Inter-Fluve recommendations were conservative and made modifications in the

criteria for Western Washington. HEC concluded that stream order was not a consistently measurable parameter and chose to use watershed area instead of stream order. HEC recommended that discharges to streams whose watershed is 100 square miles and larger at the point of discharge could be exempted from HMP requirements. They also modified the percent imperviousness criteria changing total percent impervious area (TIA) to effective impervious area (EIA). The EIA is essentially what has been termed as directly connected impervious area (DCIA). In addition to EIA they incorporate the amount of land area converted from forest to urban pervious, citing evidence that conversion from forest to urban pervious can be significant. HEC concludes that the combination of EIA and %Forest conversion (limited to less than 5%), together with minimum stream order (5th order) appears to provide an adequately conservative method to delineate exempt stream segments. HEC also studied channel gradient as a secondary criteria to exempting stream segments. They found that stream segments with a slope of 0.05% or less and that were un-restricted (i.e. no levees or entrenchment) could also be exempted. This analysis was based on the frequency of transporting salmon spawning size gravel (>16mm).

After consideration of the method and criteria used by Washington State, the Project Team concluded that the approach to using current percent imperviousness in sub-watersheds would be appropriate for the Santa Clara Basin to determine the magnitude of change from current conditions that would destabilize stable segments. In Chapter 5 of this report, the percent imperviousness of the test subwatersheds was compared to the field designated erosion classification. These results showed that as the percent imperviousness transitions from about 5% to 8%, local streams transition from stable to unstable conditions. Once this level is exceeded, the magnitude of change in impervious area allowed would be derived based on the increase in risk of erosion as a result of increase in Ep. Holding other factors such as earthquakes, rainfall patterns, and reservoir operations constant, change in Ep is associated with change in impervious area. For segments that have been destabilized as a result of activity within the watershed to date, the segment would have to be brought back to stable conditions before allowing the 3 percent increase.

7.2.2 Using criteria from the literature

There are no sub-catchments in Santa Clara Basin that are less than the 10% impervious area and are discharging to 4th or 5th order streams. Sub-catchments discharging to 4th and 5th order are in the lower portions of the watershed and have EIA in excess of 20 percent (except for subwatersheds within the riparian corridor or enclosed within parks). Therefore, the first criterion does not apply to any areas with current or future plans for development in Santa Clara Basin. There are very few stream locations where the upstream area is greater than 100 square miles. These are direct discharges in the main stem of the Coyote and Guadalupe River, downstream of confluence with Los Gatos. The direct discharge to these creeks can be exempted provided they meet the third criterion.

In Santa Clara Basin, except for some flood channels in the flat areas, stream segments that meet criterion 2 are within levees or are incised (such as main stem of Coyote and Guadalupe Creek). Based on the literature, the only unhardened channels that can be exempted are flood channels (such as Sunnyvale East and West, Wrigley Ford), provided their slopes are less

than 0.05%. Three sided hardened channels that are continuous to the tidally influenced zone can be excluded because their velocities are below Qc.

The rest of this chapter identifies portions of watersheds that cannot be exempted from hydromodification requirements, and the magnitude of change in impervious areas allowed downstream if the discharge is to stable unhardened channels outside the tidally influenced zone..

7.2.3 Most Downstream Limit (MDL) of Erosion

This section describes the basis for analyzing HMP exemptions using the MDL. As stated above, the MDL defines the point where current active erosion is observed and where any further increase in watershed development will exacerbate the observed erosion problems. Developments that discharge stormwater above this point cannot be exempted from the HMP unless other measures are taken to mitigate the expected impacts. The MDL is often bracketed at both ends with creek segments that are in a state of transition from stable to unstable conditions, except where the downstream end is hardened. Upstream of the MDL, the channels in the watershed may contain additional segments that are stable, in transition, or unstable. This section discusses the geomorphic survey methods used to delineate the MDL, presents the survey results as a GIS map, and delineates sections of the watershed above each MDL.

The Project Team obtained data from SCVWD to identify and select sections of creek that would need field investigation to locate the most downstream limit (MDL) of reach-wide erosion. The method of delineating MDLs was limited to earthen and unmodified channels with excessive reach-wide erosion (or "high" ranking as per Chapter 3) extending for lengths greater than 200 feet and ten channel widths. The geomorphologists did not look for erosion in hardened or very short channel segments between hardened sections.

Using GIS data of channel characteristics, the channel reaches were mapped to identify reaches with earthen or unmodified bottoms and banks and those with hardened bottoms and banks from concrete, sack concrete, riprap, rocks, and gabion. Typically, the lowest reaches of the watersheds are earthen, low-gradient and estuarine, the middle reaches are hardened, and the upper reaches are earthen or unmodified with short sections that are hardened. In the field, geomorphologists spot checked the hardened reaches to confirm the GIS data, and began the survey starting at the lowest section of earthen or unmodified channel in each creek. The geomorphologists walked the creeks when possible, and in areas of limited access, they spot-checked for signs of erosion at road and path crossings. Generally, they did not survey the estuarine, low-gradient reaches downstream of hardened channels. Also, typically, they did not survey above the canyon mouths[15] in headwater reaches, presuming for the purposes of this study that little channel adjustment has occurred in rural and low density residential settings.

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[15] The canyon mouth is the point where the stream transitions from the deep valley cutting into mountains to the broad floor of the greater Santa Clara Valley.

The geomorphologists located the most downstream limit of moderate to severe erosion that had occurred at an accelerated rate over an entire reach, indicating a systematic channel response to a watershed disturbance. To distinguish this from localized or historic erosion, the reach needed to exhibit pervasive and active undercutting, bank retreat, and/or incision. At this time, the distinction was qualitative in nature. The erosion had to be active and recent (within the past five years). The method used is similar to that used in Chapter 3 to classify cross-sections and segments as having stable (low), medium or high rates of erosion, with some additional measurements related to identify the length of reach with a high rate of erosion. All factors affecting channel bank and bed stability (channel slope, undercut banks, bank slope, type and density of bank vegetation, in-channel obstructions, etc.) were combined with the observed extent, estimated age, and magnitude of existing erosion to designate reach-wide erosion.

In general, the mapped reaches have bank instabilities extending along the reach a minimum of 10 channel widths and in excess of 200 feet along length of channel. Based on the technical findings that show large increases in erosive flows as a result of urbanization, the rate of erosion is expected to continue to increase with urbanization, leading to greater instability at and upstream of the MDL. The total length of unstable segments will increase upstream and potentially downstream of MDL.

7.2.4 Percent Change in Impervious Surfaces

This section describes the basis for analyzing whether channel segments downstream of MDL are at risk of becoming unstable from future development. The measure of percent imperviousness is computed as a function of the total cumulative development relative to the total watershed area upstream from the point of interest in the stream. The primary question to be addressed is "what is the increase in percent imperviousness from current conditions at which there is significant increase in risk to destabilizing stable segments". The method can also be applied upstream of current MDL once it has been rehabilitated to a stable condition.

Research in Eastern Washington determined that 5th order streams and larger could be exempt from flow controls (Inter-Fluve, 2003). Western Washington followed Eastern Washington example, although they converted the 5th order stream designation to a watershed area criteria of 100 square miles (HEC, April 2004). In the Santa Clara Basin this does not work the same as it did for Washington State because of the absence of 5th order streams. Also, there are no subwatersheds in Santa Clara Basin that are less than the 10% impervious area and are discharging to 4th or 5th order streams. Subwatersheds discharging to 4th and 5th order are in the lower portions of the watershed and have EIA in excess of 20 percent (except for subwatersheds within the riparian corridor or enclosed within parks). Also, there are very few stream locations where the upstream area is greater than 100 square miles. These are direct discharges in the main stem of the Coyote and Guadalupe River, downstream of confluence with Los Gatos.

However, what's important in the Santa Clara Basin is the change in percent imperviousness that will destabilize stable unhardened creeks. In this case, one is measuring the likelihood of potential impacts to streams from future development as a change from existing conditions. The Lower Silver - Thompson Creek subwatershed is discussed as an example

application of the approach. A similar analysis has been completed in the Berrryessa-Lower Penitencia watershed. An assessment of the full Coyote Creek watershed is nearing completion and will be incorporated in two weeks.

7.2.5 Rational for Selecting the Allowable Percent Increase in Impervious Surface Area

The above percentages specified by HEC and Inter-Fluve are based on undeveloped watersheds and would not necessarily apply to the developed and partially developed watersheds in the Santa Clara Basin. Therefore, a threshold for developing areas must be derived for the Santa Clara Basin. Ideally, an Ep analysis would be completed to accurately determine the threshold, but due to time and budget constraints an analysis of the change in impervious surfaces is used.

The current threshold for percent imperviousness is derived using two pieces of information: 1) comparison of the increase in future percent imperviousness in Thompson Creek and the associated increase in work done on the channel, and 2) the probability curve showing changes in work and associated risk of instability (Figure 5-9).

Table 7-1 lists the percent increase in impervious surface area and effective work as computed in the Thompson Creek subwatershed. This table illustrates the relationship between increases in impervious surface and work done on the stream channel. Average percent increase in work is 35%, or 2 times the increase in percent impervious surface area (the standard deviation is 0.54 and the 95% confidence limit is 3 times). This information will help determine the significance of changes in impervious surface area for other catchment areas in the Lower-Silver subwatershed and other tributaries to Coyote Creek.

Table 7-1: Percent Increase in Impervious Surface Area and Associated Erosion Potential under for Thompson Creek

Cross Section	Percent Increase in Impervious Surface Area	Percent Increase in Effective Work
TC1-6	18%	24%
TC2-1	18%	37%
TC2-3	19%	29%
TC2-6	19%	26%
TC2-9	19%	31%
TC2-10	19%	30%
TC3-5	19%	12%
TC3-7	19%	45%
TC5-2	16%	35%
TC5-3	16%	33%
TC5-4	16%	30%
TC5-5	15%	31%
TC5-6	14%	52%
TC5-7	11%	23%

The second piece of information considered is the probability curve, Ep values and risk. The percent impervious analysis applies to stream segments downstream from MDLs. These segments are currently stable and show little to no signs of active reach wide erosion. Therefore, it can be assumed that the Ep for these segments is on the order of 1 (or near 1 on the probability chart). The next question is "how much change could be allowed before the risk of channel instability becomes too high?"

As discussed previously in Chapter 5, a risk of instability of 10 percent (Ep =1.1) means that 1 in 10 streams will become unstable. A risk of 14 percent (Ep=1.2) means that 1 in 7 streams become unstable. Given a stream segment downstream of MDLs, with an Ep around 1 (say 0.9 to 1.1), no more than a 10 percent increase is acceptable. Given that the average percent increase in work is 2 times greater than the increase in impervious surfaces, the recommended threshold of impervious surface must be 50% (1/2) of the acceptable change in Ep, or 5%. Allowing for a factor of safety (and for the case when the current Ep is closer to 1.1 or 1.2, as well as the 95% confidence limit), the Project Team recommends using 3% instead of 5% increase in impervious area from current conditions as the criterion for determining exempt areas downstream from MDLs on the same creek in which the MDL is identified. The criterion can also be applied to stable segments upstream of a MDL as long as the MDL is removed by rehabilitating the creek at the MDL to a stable condition.

Creeks of higher order located downstream that drain a larger watershed order will require additional study. In Santa Clara Basin, the two major creeks that have not been evaluated for MDLs are the main stem of Coyote above 280 and Upper Penitencia. These are being studied at this time and will be reported in two weeks.

7.3 Results

Figure 7-1 shows the points that mark the downstream-most extent of erosion and the portions of the watershed upstream. Based on the locations of MDLs, the portions of the subwatersheds that are in the shaded areas will need to manage increases in erosive flows. A summary of the creeks and watersheds surveyed, the coordinates of these points, the indications of reach-wide erosion, and associated remarks for each creek are available in a GIS file that accompanies Figure 7-1.

7.3.1 Most Downstream Limit of Erosion

The results fall into four general categories:

- Several streams exhibit significant reach-wide erosion, with a distinct downstream boundary where the channel becomes hardened.

- The downstream limit of some streams is a gradation from significant erosion to low erosion characteristic of natural systems. In these streams, the earthen and unmodified reaches downstream of the downstream-most extent of reach-wide erosion (MDL) are susceptible to accelerating erosion if the storm-runoff volume, frequency and duration were to increase.

- Some streams have less than 200 feet of erosion downstream of the MDL. These earthen and unmodified reaches are likewise susceptible to increased erosion if the flow regime changes. These points have been noted as red dots on the map. Future increases in erosive flows through the MDL or the addition of impervious areas downstream of MDL can cause the MDL to migrate further downstream.

- Some streams are hardened over their entire length, with no possibility for reach-wide erosion (although their hardening may contribute to additional erosion further downstream).

Geomorphologists identified an MDL on Lower Silver Creek and each of its tributaries. The MDL on the tributaries become the MDL if an in-stream erosion protection project is implemented on Lower Silver to provide the capacity required for future flows.

On Berryessa Creek, several locations of local erosion were identified downstream of MDL, which is below the junction of Swiegert and Berryssa. These locations may transform to a reach wide problem in the future. This portion of the Berryessa watershed downstream of the MDL and the points of concern is shaded with hatched lines.

For Guadalupe Creek, the assessment was initiated at the intersection with Los Gatos Creek. It is assumed that future District projects downstream of this intersection would be designed or upgraded to include hydromodification from current and future development.

The Upper Penitencia and Coyote Creek above 280 have not been surveyed. Should MDLs be identified on these creeks, the portion of the creek at the MDL would be at risk of higher erosion and unstable conditions from future development.

7.3.2 Percent Change in Impervious Surfaces

This section summarizes the methodology for evaluating the extent of future development that could be exempted from the HMP requirements downstream of MDL, and upstream of the MDL if the creek at the MDL is rehabilitated to a stable condition.

The Lower Silver - Thompson Creek subwatershed is discussed as an example application of the approach. A similar assessment has been completed in Berryessa-Lower Penitencia Watershed. An assessment of the entire Coyote Creek watershed, which is a higher order stream, is currently being performed to determine if such restrictions also apply to a 4th order stream.

The MDL test for exemptions should be applied first to determine if a proposed development is discharging downstream of MDL. If it does, or if the MDL is rehabilitated to stable condition, the percent impervious test should be applied second to determine if the development will exceed the 3 percent threshold. If the 3 percent criterion is exceeded, the Project Team recommends that a shear stress analysis be done on stable channel segments to determine if it can be exempted in the future without upgrading the channel capacity.

Table 7-2 lists the total catchment area at points along the main stem of the Lower Silver – Thompson Creek subwatershed, impervious surface area upstream from that point for

existing conditions, and the percent imperviousness. The last two columns list the individual tributary areas and their percent of the total watershed area upstream from their junction with the main stem. Currently, the 27,810 acre watershed has 37% impervious surface area (10,159 acres). Total current percent imperviousness ranges from 20% to 37% for Lower Silver Creek. Columns 5 list the areas of each tributary. The area (acres) of change between tributary points becomes important as described in the next section.

Table 7-2. Current Impervious Surface Area for Lower Silver - Thompson Creek Subwatershed

Tributary Junctions along Main Stem	Watershed Area	Impervious Area		Tributary Area	
	(acres)	(acres)	% Imp	(acres)	% of Wtrshd
Lower Silver Mouth	27,810	10,159	37%	1,127	4.1%
Miguelita Creek	26,320	9,031	34%	2,822	10.7%
North Babb	21,852	6,209	28%	1,051	4.8%
South Babb	20,067	5,159	26%	678	3.4%
Flint Creek	16,931	4,480	26%	180	1.1%
Ruby Creek	15,532	4,301	28%	745	4.8%
Norwood Creek +Lower Silver (underground section)	14,033	3,555	25%	1,295	9.2%
Quimby Rd	11,389	2,260	20%	302	2.6%

Table 7-3 shows the allowable increase in impervious area for discharges to the segments of Lower Silver (from Quimby Road to junction with Coyote) and its tributaries, once the current MDL segments on Lower Silver and its tributaries are rehabilitated to a stable condition. MDLs have been identified on Lower Silver between Miguelita and North Babb, on South Babb and Flint Creeks. Table 7-3 includes the total catchment area and determines the allowable future increase by applying the 3 percent impervious surface area criteria. In this case, the 3 percent criterion is for discharges connected to the stream without hydromodification and accounts for the total cumulative increase in impervious surface area upstream from the point of interest. Cumulative future development in excess of this amount could potentially impact the stream segment and would be considered non-exempt.

Note again that the change experienced by the stream (column 5) is 3 percent throughout the Lower Silver - Thompson Creek subwatershed. Column 4 presents the cumulative area. All of this area could be allocated to the lowest junction, or proportionally allocated to upstream junctions such that the cumulative area is not exceeded at each junction. Else the stream will destabilize between junctions. Column 7 shows the amount of impervious area that could be added if allocated proportionately between the junctions.

7.4 Discussion

It is important to understand the significance of the MDL. The MDL is the most downstream location in the watershed where field personnel have identified *recent and active* erosion, as

opposed to erosion that may have happened in the past. Additional disturbances in the future will accelerate the rate of erosion, leading to greater channel instability.

Hydromodification is the principal contributor to active erosion in the Lower Silver – Thompson Creek, Berryessa Creek, Matadero Creek, Barron Creek, Adobe Creek, Permanente Creek, Upper San Tomas and Ross Creek subwatersheds. These subwatersheds do not have dams and gravel mines, and are primarily urban in nature.

For the creeks with dams (Stevens Creek, Los Gatos Creek, Guadalupe Creek, Alamitos Creek and Coyote Creek main stem) and mining (Calabazas Creek) future increases in development that increase Ep will cause an acceleration of erosion at and upstream from the MDL. Therefore, the impact of hydromodification has to be controlled, which implies that HMP controls are necessary for any addition of impervious surfaces upstream of MDL.

Table 7-3. Summary of Acceptable Increases in Impervious Surface Area

Tributary Junctions along Main Stem	Total Catchment Area (acres)	Allowable Increase in Impervious Area Percent	Total Cumulative Impervious Area			If allocated proportionate to area between junctions
			Current Imp Area (acres)	Allowable Future Imp Area (acres)	Increase (acres)	
Lower Silver Mouth	27,810	3.0%	10,159	10,993	834	
Miguelita Creek	26,320	3.0%	9,031	9,821	790	134
North Babb	21,852	3.0%	6,209	6,865	656	54
South Babb	20,067	3.0%	5,159	5,761	602	94
Flint Creek	16,931	3.0%	4,480	4,988	508	42
Ruby Creek	15,532	3.0%	4,301	4,767	466	45
Norwood Creek +Lower Silver (underground section)	14,033	3.0%	3,555	3,976	421	79
Quimby Rd	11,389	3.0%	2,260	2,602	342	12

It is difficult to determine without additional modeling based on cross-section measurements whether the MDL will migrate downstream in the future. Assuming that all developments upstream of MDL implement hydromodification, only those new impervious areas discharging downstream of MDL can cause an impact. However, if certain future developments upstream of MDL discharge to the creek and contribute to in creek improvements at the MDL, the unhardened segment below the MDL will also have to handle the additional flows.

The percent increase in future impervious areas can be used as an interim approach to determining if future development could potentially impact stream segments downstream from MDLs. Future increase in impervious area of 3 percent above current conditions is allowed for discharge to unhardened segments downstream of MDL. In the example shown above for Thompson-Lower Silver Creek, the percent impervious surface analysis also shows the amount of development that can proceed without hydromodification controls upstream after the MDL segments on Lower Silver and its tributaries are rehabilitated to stable conditions.

8 Discussion of Management Strategies and Implementation Requirements

This chapter provides a discussion of various issues regarding management strategies and implementation requirements. Several management strategies have been tested for the District and for the SCVURPPP. This chapter summarizes the relevant topics and conclusions from these tests. Appendix B includes a copy of the relevant technical memorandums.

8.1 Volume Control & Hydrograph Matching

Several different methods were tested and evaluated for effectiveness at meeting the erosion potential objectives when compared to the results obtained by flow duration matching. Technical Memorandum (TM) #5 (Appendix B) prepared for the SCVURPPP HMP Work Group provided a discussion on the effectiveness of volume control to manage hydromodification. Volume control means that only the amount of runoff generated from the pre-project site may be discharged from the site after development. It was recognized that volume control does not account for the differences in erosive power for the same excess volume at higher flows as compared to lower flows. To evaluate the effect of hydrograph shape while maintaining pre-project volume, the work done by several hydrographs of different shape but the same volume was compared to the work done by the pre-project hydrograph. Referring back to the work index in Chapter 5, the reader will notice that work is a non-linear function of excess shear stress. This non-linearity results in these differences in erosive power between different flow rates. The conclusion was that hydrograph shape does matter and unexpected stream erosion is likely if managing volume alone.

From this result, hydrograph matching was then tested. TM #7 prepared for the SCVURPPP HMP Work Group provided a discussion on the effectiveness of hydrograph matching. This analysis compared the effectiveness of a basin designed by matching discrete storm hydrographs at reproducing the pre-development flow duration curve. Four control basins were designed using three independent discrete storm sizes (2-year, 10-year, and 50-year), and one sized using the full 50-year hourly record of runoff generated by the HEC-HMS hydrologic model. The analysis showed that the basins sized using discrete events do not match the pre-project flow duration curve and would result in stream channel erosion. Single event management strategies do not account for frequent low flows less than the 2-year storm, antecedent conditions, partially filled basins when the following storm arrives, or the natural variability in rainfall patterns.

8.2 Flow Duration Control

Management facilities are often designed to maintain peak flow rates at their pre-development levels to prevent increases in the frequency of flooding due to new development. Facilities that control peak flow rates usually allow the duration of flows to increase, which can cause increased erosion of the downstream system. Therefore, stream systems that require protection from hydromodification require BMP's that control the duration of geomorphically significant flows (flows capable of moving sediment). Peak flow controls alone are not adequate for erosion control. In Ontario Canada, MacRae (1996)

learned that attempts to control the 2-year discharge with no consideration for the duration of flows resulted in equally degraded streams as implementing no BMP at all. In Western Washington, the Department of Ecology is requiring a flow duration standard, where pre- and post-development flow duration curves must be matched according to specified criteria.

Flow duration control requires that the increase in surface runoff created by the installation of impervious surfaces be retained on-site. The increase in runoff volume must be maintained on-site, discharge to groundwater through infiltration (can also provide facilities that enhance evapotranspiration), and/or discharged at less than the critical flow of the stream. The flow duration approach involves routing a time series (50 years of record in this case) of stormwater runoff from future development through a flow control basin that diverts and retains a certain portion of the runoff. This portion to be retained is the increase in surface runoff volume created between a pre-project and post-project development condition.

The flow duration basin is designed to have two pools a low flow pool and a high flow pool. The low flow pool is designed to capture and retain small to moderate size storms, the initial portions of larger storms, and dry weather flows. The high flow pool is designed to store and release higher flows to maintain, to the extent possible, the pre-project runoff durations.

Separate flow duration and infiltration basins may be needed for water quality reasons or a by-pass pipeline could carry the excess runoff to a safe discharge location, if feasible. Combining flow duration and infiltration into a single facility reduces the overall land requirements for stormwater management. The flow duration basin can also serves as a water quality treatment facility and can be designed to treat dry and wet weather flows using a combination of extended detention and natural treatment processes. Dry weather "nuisance flows" will also be captured and infiltrated in the basin.

8.3 Measuring the Goodness-of-Fit for Flow Duration Curves

Measuring the goodness-of-fit between the pre-project and the post-project curves is required so that planners, engineers and agency personnel can evaluate the adequacy of the flow duration basin design. In most cases, a developer will be evaluating project runoff and not flows in the receiving stream. Work cannot be evaluated without in-stream flows and cross section data. The goodness-of-fit must therefore be based on the individual flow Bins (histogram) and overall runoff volume.

One alternative is to specify a limit on the individual flow Bins of the flow duration histogram plus a limit on the overall stormwater runoff volume. For example, the post-project flow duration should not exceed 20% of the pre-project duration for all flow Bins. In addition, the overall post-project runoff volume should not exceed the overall pre-project runoff volume (i.e., zero percent increase). The overall volume is computed as the sum of the individual volumes for each of the flow Bins.

As discussed in Chapter 5, there are several reasons why the management objective for HMP could be specified as maintaining an Ep = 1. Under this strategy, the post-project flow duration must not exceed the pre-project flow duration for all points along the flow duration

curve. Over control where the post-project with BMP reduces the flow duration below the pre-project condition is acceptable in partially developed urban watersheds.

The goodness-of-fit depends on several factors; acceptable risks of instability, the HMP management objective (e.g., Ep=1), and a relationship between work and runoff volume. The goodness-of-fit can also be dependent on the ability of the developers to implement the flow duration standard. There may be space limitations, soil infiltration limitations, or other design and construction limitations. When the developer is not able to match the pre-project flow duration curve, the developer must work with the District to find a means to mitigate the residual work.

If a developer is only able to match a portion of the curve, the residual increase in runoff volume and associated work must be managed using another approach, such as in-stream upgrades. For example, if a developer only matches the curve up to the pre-project 10-year peak flow, then a residual work of 20% (i.e., Ep=1.2) must be managed by in-stream solutions or other methods.

8.4 Range of Storms to Manage

Under Provision C.3.f.iv.3 of the permit, the HMP shall identify the maximum rainfall event below which the standard applies, or range of rainfall events for which the standard applies. This section provides a discussion of evaluating the range of flows that are the most important when considering stream channel erosion and hydromodification impacts.

Cumulative work curves such as those presented in TM #4 (Appendix B) shows the undeveloped land use condition and the proposed development without any flow controls in-place. In the example illustrated in TM#4, the total work done for the undeveloped condition is 940 units. Allowing an increase of 20% (i.e., specifying a management objective of the Ep ratio = 1.2) results in an increase in work up to 1128 units. If flows are uncontrolled, the post-development work done would be 10,340 units in this stream segment. Work must be matched up to a point where the remaining uncontrolled flows do not increase the total work beyond the allowable 1128 units. In this example, work is matched between Qc and the pre-project 10-year peak flow, while the higher uncontrolled flows add an increase in work of 20%.

If an Ep value of 1 was selected as the objective for management than the cumulative work curves must be matched all along the entire length, such that the final cumulative work is equal to the pre-project condition. The work done on the stream bed has to equal the pre-project or stable condition, based on the cumulative value of the work done using a hydrologic record that is representative of 50 years of rainfall. For this, the range of storms that have to be managed increases from Qc to 50 year peak flow under pre-project conditions.

Chapter 5 (Figure 5-9) shows that for Ep values above 0.8, there is some risk of destabilizing stable channel segments. For example, a risk of 10 percent (Ep=1.1) results in a risk of 1 in 10 stable segments getting destabilized in the future. At an Ep of 1.0, there is a risk of 1 in 12 (8%) stable channel segments getting destabilized in the future. The risk is minimal at Ep of 0.8 or lower. For future developments that add impervious area, it is recommended that the

Water District be cognizant of the residual risk for discharges to the creek with an Ep > 0.8. The Water District should add the requisite grade control and bank protection downstream of the discharge to manage this risk.

The Water District may select an acceptable level of risk of 5 percent (1 in 20 stable channel segments getting destabilized following development). In this instance, a development discharging from a control basin designed with an Ep of 1.2 will need some additional in-stream structures downstream to reduce the risk of channel instability to a level less than 5 percent. District staff should simultaneously consider and design these structures at the time of District review.

9 Method Limitations and Uncertainties

9.1 Physical Processes Not Accounted For

Streams are also impacted by historical land use practices such as grazing, agriculture, and mining; the development of infrastructure such as bridges, flood control and water supply facilities; and buildings located in the floodplain. Natural events such as fires, droughts, and landslides caused by seismic activity can also impact streams. Although no less important, the effects of these factors are not addressed in this document.

Several modes of failure are recognized as contributing to channel bank failure in the Thompson Creek subwatershed. The primary failure mechanism addressed by this HMP method is shear erosion along the channel bed and at the toe of the bank, which results in incision and over-steepened banks. Secondary failure mechanisms are often a function of the incised channel form and include slumping due to over-steepened slopes and/or lack of vegetation and root structures. Also, due to the flashy nature of flows in Thompson Creek, rapid reductions in water level during the recession portion of the hydrograph can also cause bank failure.

Urbanization can also change sediment sources and supplies, alter vegetation patterns and densities, and affect flood plains. The sediment load, its particle size range, input timing and longitudinal distribution contribute to the development of geomorphic surfaces and in-stream deposits that form the foundation for riparian and aquatic habitat. The episodic nature of storms and dry and wet cycles (including El Nino), fire and earth flows affect sediment characteristics of stream channels. Episodic events temporarily establish a new set of stream conditions and vegetation patterns that gradually return to the pre-event geometry over time. The stability assessment primarily addresses persistent changes in watershed runoff patterns and does not specifically address changes in sediment or vegetation patterns. However, these influences would be accounted for under the geomorphic/ historic assessment, and characterization of the watershed and stream system.

In the implementation phase of the HMP, development will likely occur in watersheds that have dams and reservoirs, or gravel mining, water diversions, etc. that can also cause channel incision and widening. The development of the HMP method was intentionally tested on subwatersheds without these factors to identify true urbanization impacts and develop control measures that address hydromodification as defined in this report. When HMP control measures are applied to development in watersheds with other stream impacting factors, the impacts associated with hydromodification is adequately mitigated. This does not mean however that the other factors will not cause channel instability and adjustment.

9.2 Analysis of Model Errors

Chapter 4 discussed the accuracy of the hydrologic model in predicting low flows and larger peak flows when compared to other methods, including measured flow data. This section discusses how these model discrepancies are likely to affect the stability assessment. The results of the HMS model tested are briefly summarized below:

- The HMS model was compared to a regional regression equation developed for predicting 2-year to 100-year peak flows form open space. The regression equation was developed for flood control purposes. The model results for the pre-urban land use scenario were shown to under-predict peak flows estimated by the regression equation.

- The HMS model was compared to high water marks measured in the field after a December 2002 storm. The continuous model accurately predicts the December 2002 peak flows for the upper reaches (mostly open space) and predicts slightly high in the lower reach (mostly urbanized), although not out-of-bounds. The regression equation also over-predicts the measured high water marks. This error has been attributed to streambed infiltration

- Measured flow data was collected for storms between January and April 2003. The modeled flow volumes were compared to the measured stream flow volumes. The continuous model accurately predicts stream flow volume in the lower reaches (urban) and over-predicts volume in the upper reaches (open space). The event-based model over-predicts the continuous model and measured data.

It should be noted that runoff volume is more important for hydromodification than peak flows. Erosion and channel adjustment is affected by the frequency, duration and volume of long-term stream flows. Infrequent peak flows have little effect on the overall hydromodification problems.

These results suggest that the continuous HMS model is over-predicting runoff volume from open space, which becomes negligible once the runoff volume from the urban area becomes significant – likely around cross-section 5-5 in Segment 5 (for existing conditions). It appears that the most significant error is in predicting runoff for the pre-urban land use scenario, since the model seems to over-predict runoff volume from open space. This is not a significant issue under existing conditions, or for the upper reach stable cross-sections where the Ep ratio would tend to cancel the errors.

Given the above discussion, the stability assessment may be under-predicting the magnitude of the erosion potential (Ep) for most cross-sections tested. In other words, the pre-urban Work Index (W) would be smaller than our current estimates, which would than increase the estimated Ep values.

Glossary

Aggradation: The deposition and accumulation of sediment that was eroded and transported from the upstream watershed, resulting in an elevated streambed.

Alluvium: Silt, sand and gravel deposited by flowing water.

Armor: A layer of rocks on the surface of a streambed that resists erosion by water flows. The rock can be naturally occurring, caused by the scour of smaller particles from high discharges, or placed by humans to stop channel erosion.

Bankfull: The flow at which a stream with a natural, self-formed floodplain just exceeds the capacity of the banks.

Bar: A sand or gravel deposit found on the bed of a stream that is often exposed during low-water periods.

Basin: An area confined by topographic divides encompassing a number of watersheds that ultimately drain to a common point. In this report, *basin* is the term used to describe the Santa Clara Basin that contains a number of watersheds that drain to the Lower South Bay. The Santa Clara Basin is designated by the USGS as Hydrologic Cataloging Unit No. 18050003.

Bedload: Medium-grained sand and coarser material such as gravel that rolls and saltates along the bottom of the stream by the flow of water.

Clay: Hydrous aluminum silicate minerals with platy structure, typically less than 1/256-mm in diameter.

Colluvium: Material deposited by gravity at the foot of a slope.

Cross-section geometry: The geometry of a channel from which you can estimate the width, depth, area, wetted perimeter, hydraulic radius and channel conveyance.

Dominant discharge: The channel-forming discharge responsible for creating the primary channel that transports the majority of the sediment load.

Flood control levees: An embankment raised to contain the floodwaters of a stream or river.

Fluvial Geomorphology: The study of forms and characteristics of streams and the physical processes that create them. The processes of hydrology, geology, and sediment supply and transport interact with the watershed and material making up the channel boundary, creating features such as terraces, floodplains, channel planform and geometry, and instream channel features, such as pools, riffles, bars, and secondary channels.

Geomorphology: The study of forms and characteristics of the earth's surface and the physical and chemical processes that affect landforms. Weathering, erosion and transport are

the fundamental geomorphic processes that break down mountains and supply sediment to stream channels.

Geomorphic reaches: Stream segments that are fairly homogenous in terms of bed and bank characteristics, as well as channel geometry.

Grade control structures: Structures installed in a stream channel intended to reduce the energy of flow by reducing the slope of the streambed. Examples include cross-vane grade control structures, weirs and sackrete terrace structures.

Gravel: Sedimentary matter having a diameter between 4.75 and 75-mm.

Hardpan: A layer of hard subsoil or clay.

Hydrology: The scientific study of the properties, distribution, and effects of water on the earth's surface, in the soil and underlying rocks, and in the atmosphere.

Hydrologic processes: The extent to which precipitation is intercepted by vegetation, infiltrates into the ground, or results in overland flow, influencing the rate and magnitude of stream flows.

Hydrographic segments: Segments of a stream that have similar water inputs. Segments are identified based on significant flow increases in the downstream direction due to confluences with major tributaries.

Hydromodification: The change in the natural watershed hydrologic processes and runoff characteristics (i.e., interception, infiltration, overland flow, interflow and groundwater flow) caused by urbanization or other land use changes that result in increased stream flows and sediment transport.

Impervious surfaces: A hard surface area that either prevents or retards the entity of water into the soil mantle. A hard surface area which causes water to run off the surface in greater quantities or at an increased rate of flow from the flow present under natural conditions prior to development. Common impervious surfaces include: roofs, roadways, walkways, driveways, parking lots, patios, concrete or asphalt paving, gravel roads, and packed earthen material.

Imbrication: A shingling effect of deposited sediments, creating a protective barrier on the streambed.

Incision: The hydrologic processes of stream flow that exceeds the available sediment load and erodes streambeds, resulting in a deepening channel.

Knickpoint: The point of a stream bed where there is an abrupt change in slope, governed by regimen and by the structure and composition of the bed and bank materials of the river.

Loam: Soil composed of a mixture of sand, clay, silt, and organic matter.

Meander Bends: A bend in the course of a stream, developed through lateral shifting of its course toward the convex side of the bend.

Overland flow: Sheets of surface runoff created by excess precipitation that is not infiltrated or intercepted by vegetation.

Physiography: The study of the physical features of the earth's surface.

Planform: Stream channel patterns (braided, straight or meandering).

Pool: A location in an active stream channel, typically located in the outside bend of a meander, that exhibits relatively deep waters and reduced flow velocities.

Pool-riffle morphology: Alternating, regularly shaped deep and shallow areas of a stream.

Shale: A fissile rock composed of layers of claylike, fine-grained sediments.

Silt: particles with diameters between 0.75 and 0.002-mm.

Siltation: The settling of soil and sedimentary particles in lakes, rivers and streams.

Sinuosity: The ratio of stream length to valley length.

Stratigraphy: The study of rock strata, including the distribution, deposition, and age of sedimentary rocks.

Subwatershed: The catchment area of a stream tributary or series of stream tributaries.

Tributary: A stream or river flowing into a larger body of water.

Urbanization: The transformation of land into residential, commercial, and industrial properties and associated infrastructure such as drainages, roads, and sewers.

Watershed: A region drained by a single major river system.

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Table 3-1: Observed Geomorphic Reaches in the Thompson Creek Subwatershed

Code	Geomorphic Reach	Hydrographic Segment	Affected Cross-Sections[1]	Bed	Banks[2]	Remarks
A	Exposed Clay Hardpan w/ Localized Zones of Deposition	Lower Segment TC1	TC1-1 to TC1-5	Historically Incised	Localized Active Erosion	Several in-stream structures control the pattern of erosion and deposition in the reach. Incision has occurred down to the clay hardpan and has been effectively halted.
B	Historically Incised w/ Severe Bank Erosion	Upper Segment TC1 - Lower Segment TC2	TC1-6 to TC2-3	Historically Incised	Actively Eroding	Steep, vertical banks greater than 10 feet high characterize this reach that has experienced significant incision, bank erosion is severe.
C	Historically Incised w/ Bank Erosion at Bends, Sinuous Planform	Upper Segment TC2 - Lower Segment TC4	TC2-5 to TC3-7	Historically Incised	Actively Eroding	Similar to Geomorphic Reach B, but slightly more stable due to increased density of bank vegetation. Also, increased sinuosity of channel planform with excessive erosion focused at bends.
D	Channel Form Controlled by Dense Vine Vegetation	Upper Segment TC4 - Lower Segment TC5	None	Not Currently Incising	No Excessive Erosion	Extremely dense vine (blackberry) and shrub vegetation controls the stability of the reach where high flows are routed and dispersed over the dense vegetation rather than through the channel, limiting bank erosion.
E	Actively Incising w/ Bank Erosion	Mid Segment TC5	TC5-1 to TC5-5	Actively Incising	Actively Eroding	Located just downstream from some the most recent development, the channel bed in this reach is showing signs of past and present incision (exposed root mats, small knickpoints), which decreases in the upstream direction.
F	Upper End of Development, Meta-Stable, Less Disturbed	Upper Segment TC5	TC5-6 to TC5-7	Not Currently Incising	No Excessive Erosion	Located upstream from most development, this reach has relatively stable channel bed and banks, but could be impacted by a) future development and/or b) downstream incision.
G	The Gorge Reach:Canyon-like Channel w/ Landslides	Up to the headwaters "bowl"	None	Not Currently Incising	No Excessive Erosion	Periodic natural landslides probably recur, likely causing pulses of sedimentation downstream.
	Tributaries					
H	Actively Incising w/ Bank Erosion and Large Knickpoints	Segment YB1, Yerba Buena Creek	YB1-1 to YB1-7	Actively Incising	Actively Eroding	At least three large (> 5 feet) knickpoints are present along the length of Yerba Buena Creek, with some sections exhibiting very steep, canyon-like channels. Yerba Buena Creek above Villa Vista is currently stable (not incised).
I	Stable Rock- or Clay-cut Tributary Channels	Upper Evergreen Creek, Upper Yerba Buena	None	Not Currently Incising	No Excessive Erosion	Small channels cut into bedrock or hard clay with thin, immature floodplains characterize the upper sections of these tributaries, which have the potential to be impacted by downstream incision or nearby development.
J	Engineered Channels or Storm Drains	Norwood, Quimby, Fowler Creeks	None	Not Currently Incising	No Excessive Erosion	Man-made channels or pipes that convey flows to Thompson Creek that are presently stable but could be affected by incision on the main stem.

Notes:

[1] Cross-sections represent one site within a geomorphic reach and may not exhibit all characteristics that are described for the reach.

[2] Some amount of bank erosion is expected in un-disturbed stream systems, therefore, it is noted in reaches that are not experiencing severe erosion that "no excessive erosion" is occurring to distinguish natural erosion processes from accelerated erosion due to disturbance.

Table R3-1. Observed geomorphic reaches in the Ross Creek subwatershed

Code	Geomorphic Reach	Affected Cross-Sections[1]	Bed	Banks[2]	Remarks
A	Earthen Trapezoid Channel with Meandering Low Flow Channel and Inchannel "Bench"	RC1-2	Incised Since 1955	No Excessive Erosion	Channelized in 1955, channel has incised through altered bed creating a very prominent, linear "bench", has a meandering low flow channel and pool/riffle sequences, semi-stable
B	Earthen Trapezoid Channel with Steep Channel Banks and Severe Bank Erosion (Widening)	RC3	Incised Since 1955	Acitvely Eroding	Severe bank erosion, steep bank slopes, not much evidence of the "bench" seen downstream, bed is much wider than in Reaches A and C
C	Earthen Trapezoid Channel Punctuated with Grade Control Structures	RC4-6	Incised Since 1955	Acitvely Eroding	Channelized in 1957, channel has incised through altered bed creating a very prominent, linear "bench", reach is punctuated by grade control structures, localized bed and bank erosion dictated by in-stream structures
D	Non-engineered Channel with Numerous Bank Protection Projects, Low Erosion	RC7	Minor Active Incision	No Excessive Erosion	Unchannelized upper section of creek, numerous bank protection projects, trees located on lower banks, some undercutting but tree roots act as a stabilizing factor
E	Non-engineered Channel with Active Bank Widening	RC8	Minor Active Incision	Acitvely Eroding	Active bank widening, exposed tree roots, loss of vegetation, localized erosion
	East Ross Creek Tributary				
F_1	Active Incision and Bank Erosion in Lower Reach	ER1-2	Actively Incising	Acitvely Eroding	Some alteration (widening) has been conducted on the channel for flood control purposes, active incision and bank erosion, some pipes exposed up to 1.5 feet, steep banks
F_2	Stable Upper Reach of Tributary	ER3	Not Currently Incising	No Excessive Erosion	Stable channel with meandering, vegetated banks, low erosion, debris jam

Notes:

[1] Cross sections represent one site within a geomorphic reach and may not exhibit all characteristics that are described for the reach.

[2] Some amount of bank erosion is expected in undisturbed stream systems, therefore, it is noted in reaches that are not experiencing severe erosion that "no excessive erosion" is occurring to distinguish natural erosion processes from accelerated erosion due to disturbance.

Table 3-2: Hydraulic and Geomorphic Characteristics of Reaches in the Thompson Creek Subwatershed

Cross-section	Top of Bank Width	Average Depth from TOB[1]	W/D Ratio[2]	Channel Slope	D^{50} [3]	Exposed Clay Hardpan on Bed?	Depth of Mobile Material	Weakest Bank Material (left bank)[4]	Weakest Bank Material (right bank)[4]	Erosion Ranking (H,M,L)[5]
	(feet)	*(feet)*	*(unitless)*	*(feet/feet)*	*(mm)*	*(Y/N)*	*(in)*			
Segment TC1										
TC1-1	16.8	4.3	3.9	0.0016	5.8	N	24	sandy gravel	silty loam and gravel	L
TC1-2	20.3	4.7	4.3	0.0040	-	Y	3-4	silt loam	sand and small gravel	L
TC1-3	23.5	10.9	2.2	0.0064	0.0082	Y	0	pebbly clay	pebbly clay	M
TC1-4	22.8	6.1	3.7	0.0170	-	Y	6	pebbly clay	pebbly clay	M
TC1-5	25.8	9.5	2.7	0.0170	-	Y	0-24	sandy clay	silty layer	H
TC1-6	52.0	11.5	4.5	0.0102	4.5	N	12-24	sandy loam with gravel	gravelly sand	M
TC1-7	11.1	3.4	3.2	0.0091	-	N	12	silt loam with gravel/sand	silty loam with gravel/sand	M
Segment TC2										
TC2-1	46.2	13.5	3.4	0.0078	-	N	18	loamy clay	loamy clay	M-H
TC2-2	36.8	12.5	3.0	0.0160	-	N	10	clayey sand-gravel	clayey sand-gravel	L-M
TC2-3	45.4	13.2	3.5	0.0109	-	N	12	clayey sand-gravel	clayey sand-gravel	H
TC2-5	15.2	3.9	3.9	0.0117	4.0	N	> 24	silt, sand and some clay	sandy silt	M
TC2-6	30.0	7.4	4.0	0.0150	-	N	10	well sorted med sand	clayey sand and gravel	M
TC2-8	34.8	10.0	3.5	0.0013	-	N	6	sand and gravel	fine sand	H
TC2-9	15.8	4.4	3.6	-	-	N	7	colluvium	fine sand w/ some clay	L
TC2-10	59.4	13.2	4.5	0.0064	3.2	N	-	sand-gravel-silt w/ clay	sand-gravel w/ some clay	M
Segment TC3										
TC3-1	17.5	4.1	4.3	0.0032	-	N	24	sand	silty loam	L-M
TC3-2	23.7	4.2	5.6	0.0031	-	N	24	silty-sandy loam	silty loam w/ pebbles	L
TC3-3	22.9	6.3	3.7	0.0045	5.0	N	24	silty loam w/ gravel	silty fine gravelly loam	L-M
TC3-4	18.7	7.0	2.7	0.0084	-	N	24	clayey pebbly with silt loam	sandy gravelly loam	M
TC3-5	22.4	4.2	5.3	0.0099	-	N	-	silty sandy loam	gravelly silty loam	H
TC3-6	10.3	3.2	3.3	0.0086	0.0065	Y	24	sandy gravelly loam	gravelly loam	H
TC3-7	13.6	3.6	3.8	0.0094	6.9	N	18	silty loam with some gravels	silty loam with some gravels	L
Segment TC5										
TC5-1	46.1	11.9	3.9	0.0250	-	N	9.6	sand and gravel with silt	silty sand w/ some gravel	H
TC5-2	13.8	5.2	2.7	0.0058	9.0	N	>24	silty sand and gravel	silty sand and gravel	M
TC5-3	11.0	7.3	1.5	0.0047	-	N	12	silts and sands	fine sand and silt	H
TC5-4	13.9	4.4	3.2	0.0166	-	N	>24	clayey sand and gravel	sandy silt	H
TC5-5	11.7	2.3	5.2	0.0102	3.7	N	>>24	pebbly sand and silt	sand, pebbly sand	M-H
TC5-6	19.0	3.6	5.2	0.0086	8.0	N	12	well sorted sand	sand and gravel	S
TC5-7	14.0	1.8	7.8	0.0129	3.5	N	>24	clayey sand and gravel	clayey sand and gravel	S
Segment YB1										
YB1-0	4.4	0.9	4.7	0.0222	-	N	24	pebbly silty loam	pebbly silty loam	S
YB1-1	4.9	2.2	2.3	0.0179	10.2	N	12	silty pebbly loam w/ gravels	silty pebbly loam w/ gravels	M
YB1-2	7.9	2.2	3.5	0.0282	-	N	12	silty pebbly loam	silty pebbly loam	M
YB1-3	8.9	2.9	3.1	0.0221	-	N	24	silty pebbly loam w gravels	silty pebbly loam	M
YB1-4	8.2	1.6	5.1	0.0111	-	N	24	silty loam -- gravelly cobbly sand	gravelly cobbly loam	L
YB1-5	unsurveyed			-	-	Y	12	cobbly, silty, pebbly loam	cobbly, silty, pebbly loam	L
YB1-6	10.8	9.4	1.2	0.0139	0.028	Y	12	clay-silty gravelly-cobbly loam	clayey-gravelly loam, cobbly silty loam	H
YB1-7	27.7	2.5	11.0	0.0149	4.8	N	24	silty gravelly loam	silty gravelly loam	M

Notes:

[1] TOB = Top of Bank

[2] W/D ratio for full channel dimensions to TOB, not bankfull

[3] D^{50} = the median diameter of bed material determined from a samples collected from the bed surface.

[4] left/right bank = described from the observer facing downstream

[5] Erosion Ranking: S=stable, L = low, M = medium, H = high; based on bank, cross-section, and reach-wide observations/measurements

Table R3-2. Hydraulic and Geomorphic Characteristics of Cross Sections in Ross Creek, Santa Clara County, California

Cross Section	Top of Bank Width	Average Depth from TOB[1]	W/D Ratio	Channel Slope	"n" [2]	D-50 [3,4]	Depth of Mobile Material	Weakest Bank Material	Primary Erosion Mechanism (Sh, U, Sl)[5]	Local or Reach-wide erosion?	Active Incision?	Active Aggradation?	Erosion Ranking (H,M,L)[6]
	(feet)	*(feet)*	*(unitless)*	*(feet/feet)*	*(unitless)*	*(mm)*	*(in)*						
Ross Creek													
RC-1	37.0	8.6	4.3	0.0039	0.032	*15*	8	fine sand/silt	Sh	Reach-wide	N	N	M/L
RC-2	39.0	9.2	4.2	0.0067	0.036	--	14	gravel in matrix of sandy silt	Sh, U	Reach-wide	N	N	M/L
RC-3	38.4	9.3	4.1	0.0061	0.039	--	16	sandy gravel	Sh, U, Sl	Reach-wide	N	N	H
RC-4	37.5	9.2	4.1	0.0060	0.030	*20*	7	slightly cohesive sand/silt/clay	Sh, U	Reach-wide	2.0'	N	M
RC-5	41.0	10.4	3.9	0.0049	0.032	--	16	cobbles in a sandy/silt matrix	Sh, U, Sl	Reach-wide	Unsure	N	M
RC-6	33.7	9.4	3.6	0.0039	0.032	--	18	cobbles in a sand/clay matrix	Sh	Local	N	N	M/L
RC-7	25.7	6.0	4.3	0.0145	0.046	**30.4**	6	sandy clay with some gravels	U (minor)	Local	N	N	L
RC-8	24.9	6.6	3.8	0.0078	0.041	*14*	>12	sand/silt with some gravels/cobbles	Sl	Reach-wide	0.5	N	H
East Ross Creek													
ERC-1	19.3	6.5	3.0	0.0098	0.033	*23*	--	cohesive sand/silt/clay	Sh, U, Sl	Reach-wide	1'-2'	N	H
ERC-2	15.1	3.6	4.1	0.0187	0.048	--	--	cohesive clays/sand/gravels	Sh, U	Reach-wide	0.5'-1.5'	N	M/H
ERC-3	4.3	0.7	6.4	0.0191	0.045	*1.7*	6	semi-cohesive clay/silt/gravels	none	--	N	0.5'-0.75' (d/s)	L

Notes:

[1] TOB = Top of Bank

[2] Channel roughness or Manning's "n" value using the method presented by Cowan (1956)

[3] D-50 = the median diameter of bed material calculated from a sample/count

[4] D-50 in *italics* indicates analysis by volume (bed core), where as a D-50 in **bold** indicates analysis by area (Wolman pebble count)

[5] Primary Erosion Mechanism: Sh = bank shear, U = Undercutting, Sl = bank slumping

[6] Erosion Ranking: L = low, M = medium, H = high; based on bank, cross-section, and reach-wide observations/measurements

Table S3-2. Hydraulic and Geomorphic Characteristics of Cross Sections in San Tomas Creek, Santa Clara County, California

Cross Section	Top of Bank Width	Average Depth from TOB[1]	W/D Ratio	Channel Slope	"n" [2]	D-50 [3,4]	Depth of Mobile Material	Weakest Bank Material	Primary Erosion Mechanism (Sh, U, Sl)[5]	Local or Reach-wide erosion?	Active Incision?	Active Aggradation?	Erosion Ranking (H,M,L)[6]
	(feet)	*(feet)*	*(unitless)*	*(feet/feet)*	*(unitless)*	*(mm)*	*(in)*						
San Tomas Creek													
STC-1	31.9	9.8	3.2	0.0008	0.030	*13*	>18	sandy clay w/ cobbles/gravels	Sh, U	Reach-wide	2'	N	H
STC-2	37.8	11.4	3.3	0.0036	0.039	--	>12	gravel in a sandy/silty/clay matrix	Sh, U	Reach-wide	N	N	H
STC-3	21.4	2.9	7.3	0.0066	0.039	*14*	--	unconsolidated sand/silt/soil	U	Reach-wide	N	unsure	L
STC-4	18.6	1.7	10.7	0.0153	0.039	--	12	silty material some gravels	none	--	N	N	L
STC-5	19.2	6.2	3.1	0.0012	0.042	--	17	sandy clay-gravelly layers	Sh, U	Reach-wide	>1.0'	N	H
STC-6	22.0	4.9	4.5	0.0119	0.042	*22*	>12	unconsolidated silty colluvium/soil	Sl, U, Sh	Reach-wide	N	N	M
STC-7	12.2	2.1	5.7	0.0210	0.044	--	9	sandy matrix with graves/cobbles	U, Sh	Reach-wide	0.5'	N	M
STC-8	25.4	5.1	5.0	0.0099	0.044	--	12	clay/silt loam with gravels/cobbles	U	Local	0-0.5'	local	M
STC-9	19.0	7.7	2.5	0.0196	--	**25.8**	14	fine gravels	Sl, Sh, U	Reach-wide	0.5'-2'	N	H
STC-10	14.7	4.3	3.4	0.0255	0.040	**27.2**	>12	sandy matrix with graves/cobbles	Sl, Sh	Reach-wide	N	N	M/H
STC-11	13.2	2.0	6.5	0.0327	0.042	**32.4**	2	gravelly/silty sand	none	--	N	N	L
STC-12	--	--	--	0.0856	0.045	--	--	sandy matrix with graves/cobbles	U	Local	N	N	L

Notes:

[1] TOB = Top of Bank

[2] Channel roughness or Manning's "n" value using the method presented by Cowan (1956)

[3] D-50 = the median diameter of bed material determined from a sample/count

[4] D-50 in *italics* indicates analysis by volume (bed core), where as a D-50 in **bold** indicates analysis by area (Wolman pebble count)

[5] Primary Erosion Mechanism: Sh = bank shear, U = Undercutting, Sl = bank slumping

[6] Erosion Ranking: L = low, M = medium, H = high; based on bank, cross-section, and reach-wide observations/measurements

Table 3-3: Grain Size Data for Stream Bed and Bedload Material, Thompson Creek Subwatershed

Sample Location[1, 2]	Cross-sections with similar bed material [3]	D-Size[4]				Percent Gravel	Percent Sand	Percent Fines	
		D-10	*D-30*	*D-50*	*D-60*				
Unconsolidated Bed Material									
TC1-1	TC1-1	0.616	1.98	5.8	8.26	53.9	44.7	1.4	
TC1-6	TC1-5 to TC2-3	0.704	1.82	4.5	6.96	48.7	50.4	0.9	
200' u/s TC2-5	TC2-5 to TC2-8	0.581	1.56	4.0	6.62	46.5	53.2	0.3	
50' d/s TC2-10	TC2-9 to TC2-10	0.541	1.33	3.2	5.37	42.8	56.9	0.3	
TC3-3	TC3-1 to TC3-3	0.677	2.38	5.0	6.63	51.7	46.7	1.6	
TC3-7	TC3-4 to TC3-7	1.09	3.31	6.9	8.91	60.4	38.9	0.7	
TC5-2	TC5-1 to TC5-3	1.18	3.82	9.0	13	65.4	34.2	0.4	
TC5-5	TC5-4 to TC5-5	0.858	1.93	3.7	5.56	43.8	55.8	0.4	
TC5-6	TC5-6	1.17	3.98	8.0	10.8	65.6	34	0.4	
50' u/s TC5-7	TC5-7	0.236	1.12	3.5	5.8	44.2	51.6	4.2	
YB1-7	YB1-1 to YB1-2	0.443	1.67	4.8	7	49.8	49.2	1.0	
YB1-1	YB1-3, 1-4, 1-7, and 1-0	0.724	4.92	10.2	15.1	70.6	28.1	1.3	
Cohesive Bed Material								*% Silt*	*%Clay*
TC1-3	TC1-2 to TC1-3	--	0.0016	0.0082	0.021	2.7	23.6	41.1	32.6
TC3-6	Small exposure at base of bank	--	0.0015	0.0065	0.011	2.3	19	45.9	32.8
YB1-6	YB1-5 and YB1-6	--	0.0048	0.028	0.0448	0.1	29.7	46.4	23.8
Bedload[5]								Percent Fines	
TCQR	--	0.373	1.02	1.9	2.55	22	76.7	1.3	
YBSF	--	0.364	2.91	8.0	10.1	63.6	34.6	1.8	

[1] Samples were sieved at Cooper Testing Lab. Hydrometer analysis was performed on the cohesive bed material.

[2] Locations are listed from downstream to upstream within each category (unconsolidated, etc.). Segment YB1 enters Thompson Creek between segments TC4 and TC5.

[3] This column lists cross-section that have similar bed characteristics to the listed sample. These designations will be used to generalize bed conditions in the stability model.

[4] Size, in millimeters, for which 10, 30, 50, and 60 percent of the sample is finer.

[5] Bedload samples were collected at high flows during the storm on December 17, 2002. TCQR is 200 feet u/s of TC1-2; YBSF is near YB1-7. These samples were sieved at the lab and included to compare bedload (transport) to bed material (storage).

Table R3-3. Grain size data for stream bed material, Ross Creek and East Ross Creek, Santa Clara County, California

Sample Location[1]	Cross Sections with Similar Bed Material[4]	D-Size[5]				Percent > 3"	Percent Gravel	Percent Sand	Percent Silt and Clay
		D-10	*D-30*	*D-50*	*D-60*				
Ross Creek									
RC-1[2]	RC-2, RC-3	1.8	7.3	15.0	23.8	0.0	76.8	23.1	0.1
RC-4[2]	RC-5	2.5	8.9	20.0	29.1	0.0	81.5	18.4	0.1
RC-7[3]		5.0		30.4		0.0			
RC-8[2]	RC-6	2.1	7.0	15.0	23.3	0.0	78.0	21.6	0.4
East Ross Creek									
ERC-1[2]	ERC-2	1.8	10.9	23.0	29.0	0.0	82.3	17.1	0.6
ERC-3[2]		0.2	0.7	1.7	3.3	0.0	33.0	62.3	4.7

Notes:

[1] Locations are listed from downstream to upstream within each sub-watershed

[2] Samples were collected using a bed core method (volume) and then sieved at Cooper Testing Lab

[3] Analyzed by the pebble count method (area).

[4] This column lists cross-sections that have similar bed characteristics to the listed sample. These designations will be used to generalize bed conditions in the model.

[5] Size, in millimeters, for which 10, 30, 50, and 60 percent of the sample is finer ("Percentile size").

Table S3-3. Grain size data for stream bed material, San Tomas Creek, Santa Clara County, California

Sample Location[1]	Cross Sections with Similar Bed Material[4]	D-Size[5]				Percent > 3"	Percent Gravel	Percent Sand	Percent Silt and Clay
		D-10	*D-30*	*D-50*	*D-60*				
San Tomas Creek									
STC-1[2]	STC-2	2.6	6.1	13.0	20.2	0.0	76.7	23.3	0.0
STC-3[2]	STC-4	0.7	3.9	14.0	25.6	0.0	68.0	31.7	0.3
STC-6[2]	STC-5, STC-7	0.8	9.4	22.0	33.6	9.8	69.1	20.4	0.7
STC-9[3]	STC-8	1.3		25.8					
STC-10[3]		3.8		27.2					
STC-11[3]		10.7		32.4					

Notes:

[1] Locations are listed from downstream to upstream within each sub-watershed.

[2] Samples were collected using a bed core method (volume) and then sieved at Cooper Testing Lab

[3] Analyzed by the pebble count method (area).

[4] This column lists cross-sections that have similar bed characteristics to the listed sample. These designations will be used to generalize bed conditions in the model.

[5] Size, in millimeters, for which 10, 30, 50, and 60 percent of the sample is finer ("Percentile size").

Table 3-4: Measurements of Depth to Existing Channel Bed and Banks from Dated Structures at Thompson Creek

Hydrographic Segment	Geomorphic Reach	Location	Type of structure	Date of structure	Depth from base of structure to current channel bed[1] *(feet)*	Distance from structure to current bank[2] *(feet)*
TC1	A	Upstream from the Quimby Road weir	Pipe		~2.0	-
TC2	B	Quimby Creek confluence; between cross-sections TC1-7 and TC2-1	Sackrete grade control structure		2.6 - 3.0	-
TC2	C	Between cross-sections TC2-6 and TC2-8	Old weir		2.3	-
TC2	C	Between cross-sections TC2-9 and TC2-10	Pipe		1.6	-
TC5	E	Downstream from The Villages subdivision; between cross-sections TC5-2 and TC5-3	Bridge	1943	3.2	-
TC5	E	San Filipe Road crossing near The Villages subdivision; between cross-sections TC5-4 and TC5-5	Outfall structure		3.1	4.1

Notes:

[1] To gain some understanding of the amount of incision in a particular portion of Thompson Creek, depths from old structures (believed to be at grade in the past) to the current channel bed were measured. These measurements do not indicate an absolute amount of incision and should be taken as estimates of the range of incision over a length of stream. It is important to note that channel bed incision and bank erosion are usually most severe around in-channel obstructions, such as bridge abutments, outfall structures and grade control structures.

[2] Where possible, the distance from an old structure that was once even with the channel bank to the current bank was measured to estimate bank retreat. Only one of these measurements was taken for the Thompson Creek subwatershed and such measurements must be used carefully because localized bank erosion generally occurs at the contact between natural bank material and hardened bank protection.

Table 3-7: Grain Size Data for Stream Bed and Bedload Material, Ross Creek Subwatershed

Sample Location[1]	Cross-sections with Similar Bed Material[4]	D-Size[5]				Percent > 3"	Percent Gravel	Percent Sand	Percent Fines
		D-10	*D-30*	*D-50*	*D-60*				
Ross Creek									
RC-1[2]	RC-2, RC-3	1.8	7.3	15.0	23.8	0.0	76.8	23.1	0.1
RC-4[2]	RC-5	2.5	8.9	20.0	29.1	0.0	81.5	18.4	0.1
RC-7[3]		5.0		30.4		0.0			
RC-8[2]	RC-6	2.1	7.0	15.0	23.3	0.0	78.0	21.6	0.4
East Ross Creek									
ERC-1[2]	ERC-2	1.8	10.9	23.0	29.0	0.0	82.3	17.1	0.6
ERC-3[2]		0.2	0.7	1.7	3.3	0.0	33.0	62.3	4.7

Notes:

[1] Locations are listed from downstream to upstream within each sub-watershed

[2] Samples were sieved at Cooper Testing Lab

[3] Analyzed by the pebble count method

[4] This column lists cross-sections that have similar bed characteristics to the listed sample. These designations will be used to generalize bed conditions in the model.

[5] Size, in millimeters, for which 10, 30, 50, and 60 percent of the sample is finer.

Table 3-8: Hydraulic and Geomorphic Characteristics of Reaches in the San Tomas Creek Subwatershed

Cross-section	Top of Bank Width	Average Depth from TOB[1]	W/D Ratio	Channel Slope	"n" [2]	D-50 [3,4]	Depth of Mobile Material	Weakest Bank Material	Primary Erosion Mechanism (Sh, U, Sl)[5]	Local or Reach-wide erosion?	Active Incision?	Active Aggradation?	Erosion Ranking (H,M,L)[6]
	(feet)	*(feet)*	*(unitless)*	*(feet/feet)*	*(unitless)*	*(mm)*	*(in)*						
San Tomas Creek													
STC-1	31.9	9.8	3.2	0.0008	0.030	*12*	>18	sandy clay w/ cobbles/gravels	Sh, U	Reach-wide	2'	N	H
STC-2	37.8	11.4	3.3	0.0036	0.039	--	>12	gravel in a sandy/silty/clay matrix	Sh, U	Reach-wide	N	N	H
STC-3	21.4	2.9	7.3	0.0066	0.039	*13*	--	unconsolidated sand/silt/soil	U	Reach-wide	N	unsure	L
STC-4	18.6	1.7	10.7	0.0153	0.039	--	12	silty material some gravels	none	--	N	N	L
STC-5	19.2	6.2	3.1	0.0012	0.042	--	17	sandy clay-gravelly layers	Sh, U	Reach-wide	>1.0'	N	H
STC-6	22.0	4.9	4.5	0.0119	0.042	*24*	>12	unconsolidated silty colluvium/soil	Sl, U, Sh	Reach-wide	N	N	M
STC-7	12.2	2.1	5.7	0.0210	0.044	--	9	sandy matrix with graves/cobbles	U, Sh	Reach-wide	0.5'	N	M
STC-8	25.4	5.1	5.0	0.0099	0.044	--	12	clay/silt loam with gravels/cobbles	U	Local	0-0.5'	local	M
STC-9	19.0	7.7	2.5	0.0196	--	**25.8**	14	fine gravels	Sl, Sh, U	Reach-wide	0.5'-2'	N	H
STC-10	14.7	4.3	3.4	0.0255	0.040	**27.2**	>12	sandy matrix with graves/cobbles	Sl, Sh	Reach-wide	N	N	M/H
STC-11	13.2	2.0	6.5	0.0327	0.042	**32.4**	2	gravelly/silty sand	none	--	N	N	L
STC-12	--	--	--	0.0856	0.045	--	--	sandy matrix with graves/cobbles	U	Local	N	N	L

Notes:

[1] TOB = Top of Bank

[2] Channel roughness or Manning's "n" value using the method presented by Cowan (1956)

[3] D-50 = the median diameter of bed material determined from a sample/count

[4] D-50 in *italics* indicates analysis by volume (bed core), where as a D-50 in **bold** indicates analysis by area (Wolman pebble count)

[5] Primary Erosion Mechanism: Sh = bank shear, U = Undercutting, Sl = bank slumping

[6] Erosion Ranking: L = low, M = medium, H = high; based on bank, cross-section, and reach-wide observations/measurements

Table 3-5: Observed Geomorphic Reaches in the Ross Creek Subwatershed

Code	Geomorphic Reach	Affected Cross-Sections[1]	Bed	Banks[2]	Remarks
A	Earthen Trapezoid Channel with Meandering Low Flow Channel and Inchannel "Bench"	RC1-2	Historically Incised	No Excessive Erosion	Channelized in 1955, channel has incised through altered bed creating a very prominent, linear "bench", has a meandering low flow channel and pool/riffle sequences, semi-stable
B	Earthen Trapezoid Channel with Steep Channel Banks and Severe Bank Erosion (Widening)	RC3	Historically Incised	Acitvely Eroding	Severe bank erosion, steep bank slopes, not much evidence of the "bench" seen downstream, channel bed is much wider than Reaches A and C
C	Earthen Trapezoid Channel Punctuated with Grade Control Structures	RC4-6	Historically Incised	Acitvely Eroding	Channelized in 1957, channel has incised through altered bed creating a very prominent, linear "bench", reach is punctuated by grade control structures, localized bed and bank erosion dictated by in-stream structures
D	Non-engineered Channel with Numerous Bank Protection Projects, Low Erosion	RC7	Minor Active Incision	No Excessive Erosion	Unchannelized upper section of creek, numerous bank protection projects, trees located on lower banks, some undercutting but tree roots act as a stabilizing factor
E	Non-engineered Channel with Active Bank Widening	RC8	Minor Active Incision	Acitvely Eroding	Active bank widening, exposed tree roots, loss of vegetation, localized erosion
	East Ross Creek Tributary				
F_1	Active Incision and Bank Erosion in Lower Reach	ER1-2	Actively Incising	Acitvely Eroding	Some alteration (widening) has been conducted on the channel for flood control purposes, active incision and bank erosion, some pipes exposed up to 1.5 feet, steep banks
F_2	Stable Upper Reach of Tributary	ER3	Not Currently Incising	No Excessive Erosion	Stable channel with meandering, vegetated banks, low erosion, debris jam

Notes:

[1] Cross-sections represent one site within a geomorphic reach and may not exhibit all characteristics that are described for the reach.

[2] Some amount of bank erosion is expected in undisturbed stream systems, therefore, it is noted in reaches that are not experiencing severe erosion that "no excessive erosion" is occurring to distinguish natural erosion processes from accelerated erosion due to disturbance.

Table 3-6: Hydraulic and Geomorphic Characteristics of Reaches in the Ross Creek Subwatershed

Cross-section	Top of Bank Width	Average Depth from TOB[1]	W/D Ratio	Channel Slope	"n" [2]	D-50 [3,4]	Depth of Mobile Material	Weakest Bank Material	Primary Erosion Mechanism (Sh, U, Sl)[5]	Local or Reach-wide erosion?	Active Incision?	Active Aggradation?	Erosion Ranking (H,M,L)[6]
	(feet)	*(feet)*	*(unitless)*	*(feet/feet)*	*(unitless)*	*(mm)*	*(in)*						
Ross Creek													
RC-1	37.0	8.6	4.3	0.0039	0.032	*15*	8	fine sand/silt	Sh	Reach-wide	N	N	M/L
RC-2	39.0	9.2	4.2	0.0067	0.036	--	14	gravel in matrix of sandy silt	Sh, U	Reach-wide	N	N	M/L
RC-3	38.4	9.3	4.1	0.0061	0.039	--	16	sandy gravel	Sh, U, Sl	Reach-wide	N	N	H
RC-4	37.5	9.2	4.1	0.0060	0.030	*20*	7	slightly cohesive sand/silt/clay	Sh, U	Reach-wide	2.0'	N	M
RC-5	41.0	10.4	3.9	0.0049	0.032	--	16	cobbles in a sandy/silt matrix	Sh, U, Sl	Reach-wide	Unsure	N	M
RC-6	33.7	9.4	3.6	0.0039	0.032	--	18	cobbles in a sand/clay matrix	Sh	Local	N	N	M/L
RC-7	25.7	6.0	4.3	0.0145	0.046	**30.4**	6	sandy clay with some gravels	U (minor)	Local	N	N	L
RC-8	24.9	6.6	3.8	0.0078	0.041	*14*	>12	sand/silt with some gravels/cobbles	Sl	Reach-wide	0.5	N	H
East Ross Creek													
ERC-1	19.3	6.5	3.0	0.0098	0.033	*21*	--	cohesive sand/silt/clay	Sh, U, Sl	Reach-wide	1'-2'	N	H
ERC-2	15.1	3.6	4.1	0.0187	0.048	--	--	cohesive clays/sand/gravels	Sh, U	Reach-wide	0.5'-1.5'	N	M/H
ERC-3	4.3	0.7	6.4	0.0191	0.045	*1.9*	6	semi-cohesive clay/silt/gravels	none	--	N	0.5'-0.75' (d/s)	L

Notes:

[1] TOB = Top of Bank

[2] Channel roughness or Manning's "n" value using the method presented by Cowan (1956)

[3] D-50 = the median diameter of bed material determined from a sample/count

[4] D-50 in *italics* indicates analysis by volume (bed core), where as a D-50 in **bold** indicates analysis by area (Wolman pebble count)

[5] Primary Erosion Mechanism: Sh = bank shear, U = Undercutting, Sl = bank slumping

[6] Erosion Ranking: L = low, M = medium, H = high; based on bank, cross-section, and reach-wide observations/measurements

Table 3-9: Grain Size Data for Stream Bed and Bedload Material, San Tomas Creek Subwatershed

Sample Location[1]	Cross-sections with Similar Bed Material [4]	D-Size[5]				Percent > 3"	Percent Gravel	Percent Sand	Percent Fines
		D-10	*D-30*	*D-50*	*D-60*				
San Tomas Creek									
STC-1[2]		2.6	6.1	13.0	20.2	0.0	76.7	23.3	0.0
STC-3[2]		0.7	3.9	14.0	25.6	0.0	68.0	31.7	0.3
STC-6[2]	ST-5	0.8	9.4	22.0	33.6	9.8	69.1	20.4	0.7
STC-9[3]		1.3		25.8					
STC-10[3]		3.8		27.2					
STC-11[3]		10.7		32.4					

Notes:

[1] Locations are listed from downstream to upstream within each sub-watershed.

[2] Samples were sieved at Cooper Testing Lab

[3] Anylized by the pebble count method

[4] This column lists cross-sections that have similar bed characteristics to the listed sample. These designations will be used to generalize bed conditions in the model.

[5] Size, in millimeters, for which 10, 30, 50, and 60 percent of the sample is finer.

Table 4-1: Drainage Area Parameterization for SMA Modeling in HEC-HMS in the Lower Silver-Thompson Creek Subwatershed

		Area (mi^2)			Canopy Interception (in)			Surface Depression (in)			Soil Infiltration (in/hr)			Soil Profile Capacity (in)			Tension Zone Capacity (in)			Percolation (in/hr)		
Subbasin ID	Type	Existing	Future	Pre-Developed	Existing	Future	Pre-Developed	Existing	Future	Pre-Developed	Existing	Future	Pre-Developed	Existing	Future	Pre-Developed	Existing	Future	Pre-Developed	Existing	Future	Pre-Developed
0	I	0.240	0.274	0.490	0.00	0.00	0.26	0.24	0.24	0.38	0.00	0.00	0.13	0.00	0.00	11.68	0.00	0.00	4.87	0.00	0.00	0.08
	P	0.250	0.215		0.23	0.23		0.38	0.38		0.11	0.11		12.28	12.28		5.14	5.14		0.07	0.07	
1	I	0.124	0.203	0.391	0.00	0.00	0.26	0.28	0.28	0.38	0.00	0.00	0.20	0.00	0.00	12.67	0.00	0.00	5.73	0.00	0.00	0.14
	P	0.267	0.188		0.24	0.24		0.38	0.38		0.21	0.21		13.11	13.11		5.95	5.95		0.14	0.14	
2		1.260	1.260	1.260	0.24	0.19	0.24	0.38	0.36	0.38	0.25	0.20	0.25	8.31	6.80	8.31	3.23	2.56	3.23	0.17	0.13	0.17
3		1.238	1.238	1.238	0.25	0.22	0.25	0.36	0.37	0.36	0.22	0.19	0.22	8.22	7.45	8.22	3.29	2.94	3.29	0.15	0.13	0.15
4		1.069	1.069	1.069	0.24	0.22	0.24	0.37	0.37	0.37	0.16	0.15	0.16	9.29	8.73	9.29	3.79	3.53	3.79	0.11	0.10	0.11
5	I	1.524	1.724	2.195	0.00	0.00	0.16	0.32	0.32	0.38	0.00	0.00	0.25	0.00	0.00	14.87	0.00	0.00	6.66	0.00	0.00	0.17
	P	0.671	0.471		0.21	0.21		0.39	0.39		0.25	0.25		15.01	15.01		6.90	6.90		0.17	0.17	
6		0.154	0.154	--	0.22	0.19	--	0.52	0.50	--	0.07	0.06	--	6.80	6.17	--	2.86	2.50	--	0.04	0.04	--
7		0.266	0.266	0.266	0.35	0.34	0.15	0.86	0.85	0.36	0.14	0.13	0.17	12.03	11.84	15.03	5.45	5.36	6.94	0.09	0.09	0.11
8		0.882	0.882	--	0.21	0.20	--	0.41	0.41	--	0.06	0.06	--	11.92	11.53	--	4.89	4.71	--	0.04	0.04	--
9		0.030	0.030	0.030	0.23	0.17	0.15	0.37	0.37	0.38	0.11	0.08	0.12	13.97	10.78	15.14	6.45	4.85	7.02	0.07	0.05	0.08
10		1.064	1.064	1.064	0.25	0.23	0.25	0.37	0.37	0.37	0.14	0.12	0.14	10.78	9.88	10.78	4.45	4.04	4.45	0.09	0.08	0.09
11		1.429	1.429	1.429	0.24	0.23	0.26	0.37	0.37	0.38	0.11	0.10	0.12	11.65	10.79	12.10	4.84	4.45	5.04	0.07	0.07	0.08
12		1.215	1.215	1.215	0.20	0.19	0.24	0.36	0.36	0.37	0.13	0.12	0.15	8.53	8.23	9.88	3.43	3.29	4.04	0.09	0.08	0.10
13		0.253	0.253	--	0.12	0.08	--	0.37	0.37	--	0.21	0.14	--	9.29	6.55	--	3.74	2.47	--	0.14	0.09	--
14		0.222	0.222	--	0.23	0.23	--	0.37	0.37	--	0.07	0.06	--	12.46	11.99	--	5.22	5.01	--	0.04	0.04	--
15		0.840	0.840	--	0.16	0.15	--	0.35	0.35	--	0.05	0.05	--	9.59	9.06	--	3.90	3.64	--	0.03	0.03	--
16	I	0.547	0.594	0.808	0.00	0.00	0.12	0.31	0.31	0.36	0.00	0.00	0.10	0.00	0.00	16.75	0.00	0.00	6.76	0.00	0.00	0.07
	P	0.261	0.215		0.19	0.19		0.41	0.41		0.11	0.11		19.75	19.75		8.04	8.04		0.07	0.07	
17	I	1.850	1.963	2.436	0.00	0.00	0.09	0.32	0.32	0.34	0.00	0.00	0.25	0.00	0.00	13.51	0.00	0.00	5.66	0.00	0.00	0.16
	P	0.585	0.473		0.19	0.19		0.40	0.40		0.35	0.35		19.55	19.55		8.40	8.40		0.23	0.23	
18	I	3.069	3.196	3.973	0.00	0.00	0.10	0.32	0.32	0.34	0.00	0.00	0.18	0.00	0.00	12.54	0.00	0.00	5.43	0.00	0.00	0.12
	P	0.903	0.777		0.19	0.19		0.39	0.39		0.26	0.26		17.25	17.25		7.65	7.65		0.17	0.17	
19	I	0.248	0.248	0.305	0.00	0.00	0.15	0.30	0.30	0.38	0.00	0.00	0.09	0.00	0.00	15.20	0.00	0.00	7.07	0.00	0.00	0.06
	P	0.057	0.057		0.23	0.23		0.39	0.39		0.09	0.09		15.20	15.20		7.08	7.08		0.06	0.06	
20		0.295	0.295	--	0.10	0.08	--	0.39	0.39	--	0.15	0.11	--	5.87	4.69	--	2.22	1.64	--	0.10	0.07	--
21		1.747	1.747	1.747	0.24	0.21	0.24	0.36	0.37	0.38	0.23	0.20	0.23	7.55	6.69	7.55	2.98	2.59	2.98	0.16	0.14	0.16
22		3.352	3.352	3.352	0.24	0.22	0.24	0.36	0.38	0.38	0.14	0.13	0.14	10.50	10.00	10.50	4.32	4.10	4.32	0.09	0.09	0.09
23	I	0.948	1.054	1.438	0.00	0.00	0.17	0.30	0.30	0.38	0.00	0.00	0.35	0.00	0.00	15.39	0.00	0.00	6.09	0.00	0.00	0.23
	P	0.488	0.384		0.21	0.21		0.40	0.40		0.36	0.36		15.80	15.80		6.26	6.26		0.24	0.24	
24		0.258	0.258	--	0.25	0.18	--	0.38	0.38	--	0.19	0.14	--	13.94	10.29	--	5.56	3.98	--	0.12	0.09	--
25	I	0.063	0.079	0.114	0.00	0.00	0.19	0.32	0.32	0.38	0.00	0.00	0.20	0.00	0.00	12.22	0.00	0.00	5.51	0.00	0.00	0.13
	P	0.051	0.035		0.18	0.18		0.38	0.38		0.20	0.20		12.32	12.32		5.57	5.57		0.13	0.13	
26		1.046	1.046	1.046	0.25	0.18	0.25	0.38	0.36	0.38	0.19	0.14	0.19	10.13	7.55	10.13	4.09	2.94	4.09	0.13	0.09	0.13
27		1.843	1.843	1.843	0.25	0.23	0.25	0.38	0.38	0.38	0.25	0.23	0.25	6.92	6.37	6.92	2.70	2.44	2.70	0.17	0.15	0.17
28	I	0.029	0.044	0.060	0.00	0.00	0.17	0.24	0.24	0.38	0.00	0.00	0.07	0.00	0.00	13.51	0.00	0.00	5.70	0.00	0.00	0.05
	P	0.031	0.016		0.25	0.25		0.50	0.50		0.07	0.07		13.51	13.51		5.70	5.70		0.05	0.05	
29		0.789	0.789	0.789	0.25	0.22	0.25	0.38	0.37	0.38	0.25	0.23	0.25	6.92	6.32	6.92	2.70	2.42	2.70	0.17	0.15	0.17
30	I	0.015	0.030	0.040	0.00	0.00	0.18	0.24	0.24	0.38	0.00	0.00	0.08	0.00	0.00	13.55	0.00	0.00	5.73	0.00	0.00	0.05
	P	0.026	0.010		0.23	0.23		0.40	0.40		0.08	0.08		13.55	13.55		5.73	5.73		0.05	0.05	
31	I	0.041	0.064	0.248	0.00	0.00	0.22	0.24	0.24	0.38	0.00	0.00	0.20	0.00	0.00	8.79	0.00	0.00	3.55	0.00	0.00	0.14
	P	0.054	0.032		0.20	0.20		0.46	0.48		0.11	0.11		12.11	12.11		5.06	5.06		0.07	0.07	
32		0.372	0.372	0.372	0.24	0.22	0.24	0.39	0.39	0.39	0.25	0.22	0.25	6.80	6.24	6.80	2.64	2.39	2.64	0.17	0.15	0.17
33		0.597	0.597	0.597	0.25	0.23	0.25	0.38	0.37	0.38	0.25	0.23	0.25	6.92	6.32	6.92	2.69	2.42	2.69	0.17	0.15	0.17
34		0.717	0.717	0.717	0.25	0.22	0.25	0.38	0.36	0.38	0.25	0.22	0.25	6.88	6.25	6.88	2.68	2.39	2.68	0.17	0.15	0.17
35	I	0.025	0.065	0.108	0.00	0.00	0.18	0.26	0.26	0.38	0.00	0.00	0.24	0.00	0.00	7.17	0.00	0.00	2.81	0.00	0.00	0.17
	P	0.083	0.043		0.25	0.25		0.38	0.38		0.25	0.25		7.11	7.11		2.78	2.78		0.17	0.17	
36	I	0.039	0.039	0.072	0.00	0.00	0.15	0.27	0.27	0.38	0.00	0.00	0.25	0.00	0.00	6.93	0.00	0.00	2.70	0.00	0.00	0.17
	P	0.032	0.032		0.22	0.22		0.45	0.45		0.25	0.25		6.93	6.93		2.70	2.70		0.17	0.17	
37		0.153	0.153	--	0.08	0.06	--	0.34	0.35	--	0.12	0.10	--	3.77	3.43	--	1.26	1.07	--	0.08	0.07	--
38		0.267	0.267	0.153	0.23	0.20	0.23	0.37	0.35	0.37	0.23	0.20	0.23	6.49	5.80	6.49	2.50	2.18	2.50	0.16	0.14	0.16
39	I	0.136	0.185	0.267	0.00	0.00	0.15	0.29	0.29	0.38	0.00	0.00	0.15	0.00	0.00	13.58	0.00	0.00	5.77	0.00	0.00	0.10
	P	0.152	0.102		0.17	0.17		0.41	0.41		0.13	0.13		13.07	13.07		5.54	5.54		0.09	0.09	
40		0.232	0.232	0.287	0.16	0.09	0.15	0.39	0.41	0.38	0.35	0.23	0.54	12.24	7.76	17.07	4.19	2.47	6.13	0.26	0.15	0.36
41	I	0.271	0.312	0.831	0.00	0.00	0.24	0.32	0.32	0.38	0.00	0.00	0.28	0.00	0.00	12.66	0.00	0.00	4.95	0.00	0.00	0.19
	P	0.265	0.224		0.22	0.22		0.39	0.39		0.35	0.35		15.63	15.63		5.96	5.96		0.23	0.23	
42	I	0.989	1.176	1.678	0.00	0.00	0.22	0.30	0.30	0.38	0.00	0.00	0.15	0.00	0.00	13.16	0.00	0.00	5.59	0.00	0.00	0.10
	P	0.630	0.444		0.24	0.24		0.39	0.39		0.14	0.14		11.71	11.71		5.03	5.03		0.09	0.09	
43		0.047	0.047	--	0.25	0.23	--	0.37	0.37	--	0.07	0.07	--	13.02	12.01	--	5.47	5.01	--	0.05	0.04	--
44	I	1.060	1.060	1.562	0.00	0.00	0.17	0.30	0.30	0.38	0.00	0.00	0.09	0.00	0.00	13.76	0.00	0.00	5.71	0.00	0.00	0.08
	P	0.301	0.301		0.20	0.20		0.41	0.41		0.09	0.09		14.35	14.35		5.91	5.91		0.08	0.08	
45	I	0.735	0.794	2.043	0.00	0.00	0.20	0.30	0.30	0.37	0.00	0.00	0.09	0.00	0.00	18.44	0.00	0.00	7.46	0.00	0.00	0.08
	P	0.427	0.368		0.18	0.18		0.39	0.39		0.11	0.11		21.50	21.50		8.63	8.63		0.07	0.07	
46	I	0.312	0.325	0.727	0.00	0.00	0.20	0.30	0.30	0.35	0.00	0.00	0.07	0.00	0.00	12.64	0.00	0.00	5.16	0.00	0.00	0.04
	P	0.193	0.180		0.22	0.22		0.36	0.36		0.08	0.08		15.53	15.53		6.43	6.43		0.05	0.05	
47	I	1.109	1.143	2.385	0.00	0.00	0.16	0.30	0.30	0.36	0.00	0.00	0.05	0.00	0.00	12.06	0.00	0.00	4.78	0.00	0.00	0.03
	P	0.435	0.401		0.20	0.20		0.38	0.38		0.08	0.08		19.33	19.33		7.76	7.76		0.05	0.05	
48	I	1.494	1.622	2.307	0.00	0.00	0.13	0.31	0.31	0.36	0.00	0.00	0.49	0.00	0.00	19.59	0.00	0.00	8.28	0.00	0.00	0.32
	P	0.560	0.432		0.21	0.21		0.41	0.41		0.58	0.58		23.19	23.19		9.86	9.86		0.38	0.38	
49		0.234	0.234	0.232	0.23	0.19	0.26	0.37	0.36	0.38	0.21	0.16	0.24	6.83	5.97	7.43	2.85	2.25	2.93	0.14	0.12	0.16

Note:
1) I = value for impervious area, P = value for pervious area

Table 4-2: Drainage Area Parameterization for SMA Modeling in HEC-HMS in the Ross Creek Subwatershed

Subbasin ID	Type[1]	Area (mi^2)		
		Existing	Future	Pre-Urban[2]
0	I	0.150	0.150	0.223
	P	0.073	0.073	
1	I	0.432	0.432	0.657
	P	0.225	0.225	
2	I	0.656	0.656	1.042
	P	0.386	0.386	
3	I	0.570	0.570	0.897
	P	0.327	0.327	
4	I	0.683	0.683	1.205
	P	0.522	0.522	
5	I	0.622	0.622	1.060
	P	0.438	0.438	
6	I	0.184	0.184	0.394
	P	0.210	0.210	
7	E	0.546	0.546	0.546
8	I	0.220	0.220	0.494
	P	0.274	0.274	
9	I	0.107	0.107	0.195
	P	0.088	0.088	
10	E	0.150	0.150	0.150
11	I	0.097	0.097	0.173
	P	0.076	0.076	
12	R	0.810	0.810	0.810
13	I	0.148	0.148	0.331
	P	0.183	0.183	
14	R	0.600	0.600	0.600
15	R	0.191	0.191	0.191
16	R	0.473	0.473	0.473
17	R	0.180	0.180	0.180
	TOTAL:	9.620	9.620	9.620

Notes:

1) I = Impervious urban; P = Pervious urban; E = Pervious urban external of storm drain system; R = Rural

2) All pre-urban subbasins are considered rural.

Table 4-2s: Drainage Area Parameterization for SMA Modeling in HEC-HMS in the San Tomas Creek Watershed

Pre-Urban Condition:

Subbasin ID	Area (mi^2)
PRE1	2.586
PRE2	4.614
PRE3a	1.242
PRE3b	1.112
PRE3c	0.428
PRE3d	0.720
PRE3e	0.144
PRE4	2.093
PRE5	1.328
Total =	14.27

Existing Condition:

Subbasin ID	Area (mi^2)
EX1-imp	0.111
EX1-per	0.246
EX2-imp	0.123
EX2-per	0.461
EX3-imp	0.157
EX3-per	0.088
EX4a-imp	0.116
EX4a-per	0.312
EX4b-imp	0.189
EX4b-per	0.513
EX4c-imp	0.052
EX4c-per	0.140
EX5-imp	0.422
EX5-per	1.200
EX6-imp	0.327
EX6-per	1.715
EX7-imp	0.763
EX7-per	0.488
EX8-imp	0.699
EX8-per	0.486
EX9-imp	3.018
EX9-per	1.698
EX10-rur	0.969
EX11a-rur	1.242
EX11b-rur	1.112
Total =	16.64

Future Condition:

Subbasin ID	Area (mi^2)
FUT1-imp	0.268
FUT1-per	0.089
FUT2-imp	0.438
FUT2-per	0.146
FUT3-imp	0.184
FUT3-per	0.061
FUT4a-imp	0.321
FUT4a-per	0.107
FUT4b-imp	0.526
FUT4b-per	0.176
FUT4c-imp	0.144
FUT4c-per	0.048
FUT5-imp	0.973
FUT5-per	0.648
FUT6-imp	1.531
FUT6-per	0.511
FUT7-imp	0.763
FUT7-per	0.488
FUT8-imp	0.889
FUT8-per	0.296
FUT9-imp	3.536
FUT9-per	1.179
FUT10-rur	0.969
FUT11a-rur	1.242
FUT11b-rur	1.112
Total =	16.64

Notes:
1) imp = Impervious urban; per = Pervious urban; rur = Rural.
2) All pre-urban subbasins are considered rural.

Table 4-3: Drainage Area Curve Numbers and Initial Abstractions Losses

		Runoff Curve Number (CN)			*Initial Abstraction*		
Subbasin ID	**Type**	**Existing**	**Future**	**Pre-Developed**	**Existing**	**Future**	**Pre-Developed**
0	I	98.0	98.0	76.6	0.05	0.05	0.61
	P	77.8	77.8		0.57	0.57	
1	I	98.0	98.0	73.2	0.05	0.05	0.73
	P	73.3	73.3		0.73	0.73	
2		71.7	77.5	71.7	0.79	0.58	0.79
3		72.7	75.4	72.7	0.75	0.65	0.75
4		75.6	77.4	75.6	0.64	0.58	0.64
5	I	98.0	98.0	79.6	0.05	0.05	0.51
	P	73.1	73.1		0.74	0.74	
6		80.5	83.8	--	0.48	0.39	--
7		79.8	80.5	79.8	0.51	0.49	0.51
8		81.6	82.8	--	0.45	0.42	--
9		78.7	85.4	78.7	0.54	0.34	0.54
10		76.1	78.3	76.1	0.63	0.55	0.63
11		77.3	79.2	77.0	0.59	0.52	0.60
12		79.6	81.1	75.7	0.51	0.47	0.64
13		79.9	93.0	--	0.50	0.15	--
14		79.8	80.8	--	0.51	0.47	--
15		85.0	87.9	--	0.35	0.28	--
16	I	98.0	98.0	86.2	0.05	0.05	0.32
	P	79.1	79.1		0.53	0.53	
17	I	98.0	98.0	84.3	0.05	0.05	0.37
	P	70.9	70.9		0.82	0.82	
18	I	98.0	98.0	85.4	0.05	0.05	0.34
	P	73.9	73.9		0.71	0.71	
19	I	98.0	98.0	84.6	0.05	0.05	0.36
	P	78.6	78.6		0.54	0.54	
20		83.9	95.6	--	0.38	0.09	--
21		72.4	75.9	72.4	0.76	0.64	0.76
22		76.6	78.0	76.6	0.61	0.56	0.61
23	I	98.0	98.0	77.8	0.05	0.05	0.57
	P	69.4	69.4		0.88	0.88	
24		72.7	81.7	--	0.75	0.45	--
25	I	98.0	98.0	80.0	0.05	0.05	0.50
	P	76.6	76.6		0.61	0.61	
26		73.7	80.7	73.7	0.71	0.48	0.71
27		71.9	74.4	71.9	0.78	0.69	0.78
28	I	98.0	98.0	84.3	0.05	0.05	0.37
	P	79.9	79.9		0.50	0.50	
29		72.1	74.7	72.1	0.77	0.68	0.77
30	I	98.0	98.0	83.9	0.05	0.05	0.38
	P	79.2	79.2		0.52	0.52	
31	I	98.0	98.0	78.6	0.05	0.05	0.54
	P	79.8	79.8		0.51	0.51	
32		72.4	75.1	72.4	0.76	0.66	0.76
33		71.9	74.6	71.9	0.78	0.68	0.78
34		72.0	74.9	72.0	0.78	0.67	0.78
35	I	98.0	98.0	80.3	0.05	0.05	0.49
	P	71.7	71.7		0.79	0.79	
36	I	98.0	98.0	80.5	0.05	0.05	0.48
	P	73.1	73.1		0.74	0.74	
37		87.4	97.1	--	0.29	0.06	--
38		74.0	78.0	74.0	0.70	0.57	0.70
39	I	98.0	98.0	83.1	0.05	0.05	0.41
	P	79.3	79.3		0.52	0.52	
40		72.5	95.4	72.5	0.76	0.10	0.76
41	I	98.0	98.0	74.3	0.05	0.05	0.69
	P	69.3	69.3		0.88	0.88	
42	I	98.0	98.0	79.1	0.05	0.05	0.53
	P	76.8	76.8		0.60	0.60	
43		78.8	80.7	--	0.54	0.48	--
44	I	98.0	98.0	83.5	0.05	0.05	0.39
	P	79.7	79.7		0.51	0.51	
45	I	98.0	98.0	81.4	0.05	0.05	0.46
	P	79.5	79.5		0.52	0.52	
46	I	98.0	98.0	82.4	0.05	0.05	0.43
	P	79.0	79.0		0.53	0.53	
47	I	98.0	98.0	85.4	0.05	0.05	0.34
	P	80.1	80.1		0.50	0.50	
48	I	98.0	98.0	74.6	0.05	0.05	0.68
	P	62.1	62.1		1.22	1.22	
49		74.6	79.6	71.8	0.68	0.51	0.79

Table 4-4: Time of Concentration and Storage Coefficient, Thompson Creek, Pre-Urban Conditions

Subbasin	Length (ft)	Min. Elev. (ft)	Max. Elev. (ft)	Slope (ft/ft)	n	Tc (hr)	R / (Tc + R)	R
0	6600	460	800	0.0515	0.070	0.494	0.900	4.449
1	6800	320	620	0.0441	0.070	0.520	0.900	4.679
2	19200	310	2203	0.0986	0.070	0.701	0.750	2.103
3	10500	523	2449	0.1834	0.070	0.456	0.750	1.369
4	10900	480	2403	0.1764	0.070	0.469	0.750	1.406
5	11000	135	165	0.0027	0.070	1.254	0.900	11.283
7	3200	126	134	0.0025	0.070	0.716	0.900	6.446
9	2300	125	134	0.0039	0.070	0.552	0.900	4.968
10	9800	495	2418	0.1962	0.070	0.435	0.750	1.304
11	16100	259	2384	0.1320	0.070	0.603	0.750	1.808
12	10200	243	1842	0.1568	0.070	0.467	0.750	1.401
16	9400	125	620	0.0527	0.070	0.581	0.900	5.226
17	15700	80	125	0.0029	0.070	1.465	0.900	13.182
18	17200	85	130	0.0026	0.070	1.562	0.900	14.058
19	3800	135	145	0.0026	0.070	0.767	0.900	6.904
21	14700	448	2273	0.1241	0.070	0.586	0.750	1.757
22	17700	291	2269	0.1118	0.070	0.655	0.750	1.965
23	9950	220	500	0.0281	0.070	0.691	0.900	6.220
25	2400	280	460	0.0750	0.070	0.281	0.900	2.532
26	11300	487	2341	0.1641	0.070	0.485	0.750	1.454
27	12800	664	2153	0.1163	0.070	0.557	0.750	1.672
28	2500	520	600	0.0320	0.070	0.350	0.900	3.153
29	9200	845	2480	0.1777	0.070	0.432	0.750	1.296
30	1600	460	520	0.0375	0.070	0.274	0.900	2.463
31	6350	520	920	0.0630	0.070	0.463	0.900	4.167
32	8300	689	1789	0.1325	0.070	0.441	0.750	1.323
33	9400	664	2246	0.1683	0.070	0.442	0.750	1.326
34	10900	644	2491	0.1694	0.070	0.473	0.750	1.419
35	3300	560	760	0.0606	0.070	0.344	0.900	3.092
36	2550	600	780	0.0706	0.070	0.294	0.900	2.642
38	6000	604	1417	0.1355	0.070	0.377	0.750	1.130
39	5700	420	600	0.0316	0.070	0.518	0.900	4.659
40	4200	303	503	0.0476	0.070	0.407	0.900	3.665
41	11200	320	920	0.0536	0.070	0.628	0.900	5.652
42	14400	180	1200	0.0708	0.070	0.662	0.900	5.957
44	13800	135	810	0.0489	0.070	0.708	0.900	6.369
45	13700	130	1340	0.0883	0.070	0.614	0.900	5.525
46	11500	130	880	0.0652	0.070	0.607	0.900	5.464
47	9600	125	860	0.0766	0.070	0.537	0.900	4.834
48	17100	85	500	0.0243	0.070	0.923	0.900	8.306
49	4300	552	1050	0.1158	0.070	0.334	0.900	3.007

Notes:

Overland flow Manning's roughness coefficient 0.07 represents "sparse vegetation" (SCVWD, 1998).

The ratio of R / (Tc + R) lies between .5 and .9 for pervious and rural areas (SCVWD, 1998);

a ratio of 0.75 was chosen for rural hillside open space, and 0.9 for agricultural land use.

Table 4-5: Time of Concentration and Storage Coefficient, Thompson Creek, Existing Conditions, Urban Areas

					1	2	3	4						5	6			5	7					5	8	9			
sub basin i.d.	urban area (sq mi)	urban area (sq ft)	impervious %	impervious area Ai	Street area Ast	Street width Wst	Street Length Lst	Overland Impervious							Gutter				Urban Impervious Time of Concentration TCimp	urban pervious area (to s.d.)	Overland Pervious				Urban Pervious Time of Concentration TCperv	Urban: R / (TC + R)		Urban: R	
				(sq ft)	(sq ft)	(ft)	(ft)	Li (ft)	n	L	Emin	Emax	S (ft/ft)	TCI (hr)	Lcb (ft)	n	S (ft/ft)	TCcb (hr)	(hr)	(sq ft)	Lp (ft)	n	S	TCp (hr)	(hr)	Imp	Perv	Imp	Perv
0	0.490	13647501	0.490	6688161	2729500	50	54590	36.3	0.05	5900	464	822	0.061	0.035	300	0.02	0.061	0.062	0.097	6959340	63.742	0.5	0.061	0.135	0.135	0.4	0.75	0.065	0.406
1	0.391	10886769	0.318	3456747	2177354	50	43547	14.7	0.05	200	0	20	0.100	0.020	300	0.02	0.100	0.055	0.075	7430022	85.310	0.5	0.100	0.138	0.138	0.4	0.75	0.050	0.414
5	2.195	61202223	0.694	42482502	12240445	50	244809	61.8	0.05	10100	137	377	0.024	0.056	300	0.02	0.024	0.077	0.133	1.9E+07	38.233	0.5	0.024	0.133	0.133	0.4	0.75	0.089	0.398
16	0.808	22534875	0.677	15263262	4506975	50	90140	59.7	0.05	600	0	20	0.033	0.051	300	0.02	0.033	0.071	0.122	7271613	40.335	0.5	0.033	0.126	0.126	0.4	0.75	0.082	0.377
17	2.436	67902507	0.760	51588342	13580501	50	271610	70.0	0.05	15700	67	129	0.004	0.091	300	0.02	0.004	0.117	0.208	1.6E+07	30.032	0.5	0.004	0.181	0.181	0.4	0.75	0.139	0.542
18	3.973	110751723	0.773	85565556	22150345	50	443007	71.6	0.05	17100	81	250	0.010	0.074	300	0.02	0.010	0.095	0.169	2.5E+07	28.426	0.5	0.010	0.142	0.142	0.4	0.75	0.113	0.426
19	0.305	8496108	0.814	6914808	1699222	50	33984	76.7	0.05	3900	127	147	0.005	0.089	300	0.02	0.005	0.110	0.200	1581300	23.265	0.5	0.005	0.151	0.151	0.4	0.75	0.133	0.452
23	1.438	40076604	0.660	26435547	8015321	50	160306	57.5	0.05	10000	218	521	0.030	0.051	300	0.02	0.030	0.073	0.124	1.4E+07	42.547	0.5	0.030	0.132	0.132	0.4	0.75	0.083	0.395
25	0.114	3176487	0.549	1744290	635297.4	50	12706	43.6	0.05	2400	281	463	0.076	0.036	300	0.02	0.076	0.059	0.095	1432197	56.359	0.5	0.076	0.121	0.121	0.4	0.75	0.063	0.364
28	0.060	1672605	0.478	799776	334521	50	6690	34.8	0.05	2500	522	619	0.039	0.038	300	0.02	0.039	0.069	0.107	872829	65.230	0.5	0.039	0.152	0.152	0.4	0.75	0.071	0.456
30	0.040	1123074	0.361	404973	224614.8	50	4492	20.1	0.05	1600	469	547	0.049	0.028	300	0.02	0.049	0.065	0.093	718101	79.926	0.5	0.049	0.159	0.159	0.4	0.75	0.062	0.476
31	0.095	2660994	0.433	1151730	532198.8	50	10644	29.1	0.05	3000	527	631	0.035	0.036	300	0.02	0.035	0.070	0.107	1509264	70.898	0.5	0.035	0.162	0.162	0.4	0.75	0.071	0.487
35	0.108	3013380	0.235	707868	602676	50	12054	4.4	0.05	3300	543	773	0.070	0.013	300	0.02	0.070	0.060	0.072	2305512	95.636	0.5	0.070	0.159	0.159	0.4	0.75	0.048	0.476
36	0.072	2000349	0.550	1101033	400069.8	50	8001	43.8	0.05	2400	613	812	0.083	0.036	300	0.02	0.083	0.057	0.093	899316	56.197	0.5	0.083	0.119	0.119	0.4	0.75	0.062	0.356
39	0.287	8011692	0.472	3783006	1602338	50	32047	34.0	0.05	5700	406	620	0.038	0.038	300	0.02	0.038	0.069	0.107	4228686	65.977	0.5	0.038	0.154	0.154	0.4	0.75	0.072	0.462
41	0.536	14942430	0.505	7551675	2988486	50	59770	38.2	0.05	5900	318	587	0.046	0.039	300	0.02	0.046	0.066	0.105	7390755	61.827	0.5	0.046	0.143	0.143	0.4	0.75	0.070	0.428
42	1.620	45158337	0.611	27581661	9031667	50	180633	51.3	0.05	11000	181	583	0.037	0.047	300	0.02	0.037	0.070	0.116	1.8E+07	48.653	0.5	0.037	0.134	0.134	0.4	0.75	0.077	0.403
44	1.361	37948239	0.779	29550222	7589648	50	151793	72.3	0.05	12500	137	526	0.031	0.057	300	0.02	0.031	0.072	0.129	8398017	27.663	0.5	0.031	0.107	0.107	0.4	0.75	0.086	0.321
45	1.162	32392917	0.632	20481165	6478583	50	129572	54.0	0.05	8600	128	359	0.027	0.051	300	0.02	0.027	0.075	0.126	1.2E+07	45.966	0.5	0.027	0.141	0.141	0.4	0.75	0.084	0.422
46	0.505	14067450	0.618	8699733	2813490	50	56270	52.3	0.05	9100	124	477	0.039	0.046	300	0.02	0.039	0.069	0.115	5367717	47.696	0.5	0.039	0.131	0.131	0.4	0.75	0.077	0.394
47	1.544	43055568	0.718	30925926	8611114	50	172222	64.8	0.05	8400	130	650	0.062	0.046	300	0.02	0.062	0.061	0.107	1.2E+07	35.215	0.5	0.062	0.102	0.102	0.4	0.75	0.072	0.306
48	2.054	57257856	0.727	41649327	11451571	50	229031	65.9	0.05	14700	78	310	0.016	0.064	300	0.02	0.016	0.085	0.149	1.6E+07	34.075	0.5	0.016	0.138	0.138	0.4	0.75	0.099	0.415

1 Source: Nolte & Assoc 2000, Table 1 gives street area as 20% of total area for range of urban land uses
2 Source: SCVWD Hydrology Procedures, 1998. Typical average street width = 50 ft.
3 Source: SCVWD Hydrology Procedures, 1998. Lst=Ast/Wst
4 Source: SCVWD Hydrology Procedures, 1998. Li=Ai-Ast/2Lst
5 Source: SCVWD Hydrology Procedures, 1998. Kerby-Hathaway Tc equation
6 Source: Nolte & Assoc 2000, average gutter length between catch basins = 300 ft.
7 Source: SCVWD Hydrology Procedures, 1998. TCimp= TCi + TCcb
8 Source: SCVWD Hydrology Procedures, 1998. TCperv= TCimp + TCp
9 Source: SCVWD Hydrology Procedures, 1998 (p.26, range); Nolte & Assoc 2000 uses 0.4 and 0.75.

Note: All Tc and R values less than 0.1 are rounded up to 0.1 for input into HEC-HMS.

Table 4-6: Time of Concentration and Storage Coefficient, Thompson Creek, Existing Conditions, Rural and External Pervious Areas

	Subbasin	Length (ft)	Min. Elev. (ft)	Max. Elev. (ft)	Slope (ft/ft)	n		Tc (hr)	R / (Tc + R)	R
Rural	2	19200	310	2203	0.0986	0.070		0.701	0.750	2.103
Subbasins	3	10500	523	2449	0.1834	0.070		0.456	0.750	1.369
	4	10900	480	2403	0.1764	0.070		0.469	0.750	1.406
	7	3200	126	134	0.0025	0.070		0.716	0.750	2.149
	9	2300	125	134	0.0039	0.070		0.552	0.750	1.656
	10	9800	495	2418	0.1962	0.070		0.435	0.750	1.304
	11	16100	259	2384	0.1320	0.070		0.603	0.750	1.808
	12	10200	243	1842	0.1568	0.070		0.467	0.750	1.401
	21	14700	448	2273	0.1241	0.070		0.586	0.750	1.757
	22	17700	291	2269	0.1118	0.070		0.655	0.750	1.965
	26	11300	487	2341	0.1641	0.070		0.485	0.750	1.454
	27	12800	664	2153	0.1163	0.070		0.557	0.750	1.672
	29	9200	845	2480	0.1777	0.070		0.432	0.750	1.296
	32	8300	689	1789	0.1325	0.070		0.441	0.750	1.323
	33	9400	664	2246	0.1683	0.070		0.442	0.750	1.326
	34	10900	644	2491	0.1694	0.070		0.473	0.750	1.419
	38	6000	604	1417	0.1355	0.070		0.377	0.750	1.130
	40	4200	303	503	0.0476	0.070		0.407	0.750	1.222
	49	4300	552	1050	0.1158	0.070		0.334	0.750	1.002
							plus Urban Tc-Impervious			
External	6	1900	320	609	0.1521	0.070	0.129	0.343	0.750	1.028
Pervious	8	7700	246	1342	0.1423	0.070	0.126	0.545	0.750	1.634
Subbasins	13	3400	233	502	0.0791	0.070	0.149	0.476	0.750	1.428
	14	5000	279	854	0.1150	0.070	0.115	0.474	0.750	1.423
	15	3500	212	943	0.2089	0.070	0.107	0.372	0.750	1.115
	20	5200	506	920	0.0796	0.070	0.105	0.504	0.750	1.511
	24	4600	441	1182	0.1611	0.070	0.116	0.435	0.750	1.306
	37	3500	616	963	0.0991	0.070	0.107	0.421	0.750	1.264
	43	1300	520	823	0.2331	0.070	0.129	0.291	0.750	0.872

Notes:

Overland flow Manning's roughness coefficient 0.07 represents "sparse vegetation" (SCVWD, 1998).

The ratio of R / (Tc + R) lies between .5 and .9 for pervious and rural areas (SCVWD, 1998);

a ratio of 0.75 was chosen for rural hillside open space.

Table 4-7: Time of Concentration and Storage Coefficient, Thompson Creek, Future Conditions, Urban Areas

					1	2	3	4						5	6			5	7					5	8	9			
sub basin i.d.	urban area (sq mi)	urban area (sq ft)	impervious %	impervious area Ai	Street area Ast	Street width Wst	Street Length Lst	Overland Impervious							Gutter				Urban Impervious Time of Concentration TCimp	urban pervious area (to s.d.)	Overland Pervious				Urban Pervious Time of Concentration TCperv	Urban: R / (TC + R)		Urban: R	
				(sq ft)	(sq ft)	(ft)	(ft)	Li (ft)	n	L	Emin	Emax	S (ft/ft)	TCi (hr)	Lcb (ft)	n	S (ft/ft)	TCcb (hr)	(hr)	(sq ft)	Lp (ft)	n	S	TCp (hr)	(hr)	Imp	Perv	Imp	Perv
0	0.490	13647501	0.560	7648725	2729500	50	54590	45.1	0.05	5900	464	822	0.061	0.039	300	0.02	0.061	0.062	0.101	5998776	54.944	0.5	0.061	0.126	0.126	0.4	0.75	0.067	0.379
1	0.391	10886769	0.520	5656637.8	2177354	50	43547	39.9	0.05	200	0	20	0.100	0.033	300	0.02	0.100	0.055	0.088	5230131	60.051	0.5	0.100	0.117	0.117	0.4	0.75	0.058	0.351
5	2.195	61202223	0.785	48070852	12240445	50	244809	73.2	0.05	10100	137	377	0.024	0.061	300	0.02	0.024	0.077	0.138	13131371	26.820	0.5	0.024	0.112	0.112	0.4	0.75	0.092	0.337
16	0.808	22534875	0.734	16546004	4506975	50	90140	66.8	0.05	600	0	20	0.033	0.054	300	0.02	0.033	0.071	0.125	5988871	33.220	0.5	0.033	0.115	0.115	0.4	0.75	0.083	0.344
17	2.436	67902507	0.806	54711700	13580501	50	271610	75.7	0.05	15700	67	129	0.004	0.095	300	0.02	0.004	0.117	0.212	13190807	24.283	0.5	0.004	0.163	0.163	0.4	0.75	0.141	0.490
18	3.973	110751723	0.804	89089310	22150345	50	443007	75.6	0.05	17100	81	250	0.010	0.076	300	0.02	0.010	0.095	0.171	21662413	24.449	0.5	0.010	0.132	0.132	0.4	0.75	0.114	0.397
19	0.305	8496108	0.814	6914808	1699222	50	33984	76.7	0.05	3900	127	147	0.005	0.089	300	0.02	0.005	0.110	0.200	1581300	23.265	0.5	0.005	0.151	0.151	0.4	0.75	0.133	0.452
23	1.438	40076604	0.733	29383463	8015321	50	160306	66.6	0.05	10000	218	521	0.030	0.055	300	0.02	0.030	0.073	0.128	10693141	33.352	0.5	0.030	0.118	0.118	0.4	0.75	0.085	0.353
25	0.114	3176487	0.696	2211757.7	635297.4	50	12706	62.0	0.05	2400	281	463	0.076	0.043	300	0.02	0.076	0.059	0.102	964729.3	37.964	0.5	0.076	0.101	0.101	0.4	0.75	0.068	0.302
28	0.060	1672605	0.733	1225421.4	334521	50	6690	66.6	0.05	2500	522	619	0.039	0.052	300	0.02	0.039	0.069	0.121	447183.6	33.420	0.5	0.039	0.111	0.111	0.4	0.75	0.080	0.333
30	0.040	1123074	0.748	840066.15	224614.8	50	4492	68.5	0.05	1600	469	547	0.049	0.050	300	0.02	0.049	0.065	0.115	283007.8	31.499	0.5	0.049	0.102	0.102	0.4	0.75	0.077	0.307
31	0.095	2660994	0.668	1778395.8	532198.8	50	10644	58.5	0.05	3000	527	631	0.035	0.050	300	0.02	0.035	0.070	0.121	882598.2	41.460	0.5	0.035	0.126	0.126	0.4	0.75	0.080	0.378
35	0.108	3013380	0.602	1813851.5	602676	50	12054	50.2	0.05	3300	543	773	0.070	0.040	300	0.02	0.070	0.060	0.099	1199528	49.758	0.5	0.070	0.117	0.117	0.4	0.75	0.066	0.350
36	0.072	2000349	0.550	1101033	400069.8	50	8001	43.8	0.05	2400	613	812	0.083	0.036	300	0.02	0.083	0.057	0.093	899316	56.197	0.5	0.083	0.119	0.119	0.4	0.75	0.062	0.356
39	0.287	8011692	0.644	5156041.9	1602338	50	32047	55.4	0.05	5700	406	620	0.038	0.048	300	0.02	0.038	0.069	0.117	2855650	44.554	0.5	0.038	0.128	0.128	0.4	0.75	0.078	0.384
41	0.536	14942430	0.581	8685790.1	2988486	50	59770	47.7	0.05	5900	318	587	0.046	0.043	300	0.02	0.046	0.066	0.109	6256640	52.340	0.5	0.046	0.132	0.132	0.4	0.75	0.073	0.396
42	1.620	45158337	0.726	32788557	9031667	50	180633	65.8	0.05	11000	181	583	0.037	0.052	300	0.02	0.037	0.070	0.122	12369780	34.240	0.5	0.037	0.114	0.114	0.4	0.75	0.081	0.342
44	1.361	37948239	0.779	29550222	7589648	50	151793	72.3	0.05	12500	137	526	0.031	0.057	300	0.02	0.031	0.072	0.129	8398017	27.663	0.5	0.031	0.107	0.107	0.4	0.75	0.086	0.321
45	1.162	32392917	0.683	22136303	6478583	50	129572	60.4	0.05	8600	128	359	0.027	0.054	300	0.02	0.027	0.075	0.129	10256614	39.579	0.5	0.027	0.131	0.131	0.4	0.75	0.086	0.393
46	0.505	14067450	0.643	9049832.5	2813490	50	56270	55.4	0.05	9100	124	477	0.039	0.048	300	0.02	0.039	0.069	0.116	5017617	44.585	0.5	0.039	0.127	0.127	0.4	0.75	0.078	0.381
47	1.544	43055568	0.740	31873114	8611114	50	172222	67.5	0.05	8400	130	650	0.062	0.047	300	0.02	0.062	0.061	0.108	11182454	32.465	0.5	0.062	0.098	0.098	0.4	0.75	0.072	0.294
48	2.054	57257856	0.790	45208194	11451571	50	229031	73.7	0.05	14700	78	310	0.016	0.067	300	0.02	0.016	0.085	0.152	12049662	26.306	0.5	0.016	0.123	0.123	0.4	0.75	0.101	0.368

1 Source: Nolte & Assoc 2000, Table 1 gives street area as 20% of total area for range of urban land uses
2 Source: SCVWD Hydrology Procedures, 1998. Typical average street width = 50 ft.
3 Source: SCVWD Hydrology Procedures, 1998. Lst=Ast/Wst
4 Source: SCVWD Hydrology Procedures, 1998. Li=Ai-Ast/2Lst
5 Source: SCVWD Hydrology Procedures, 1998. Kerby-Hathaway Tc equation
6 Source: Nolte & Assoc 2000, average gutter length between catch basins = 300 ft.
7 Source: SCVWD Hydrology Procedures, 1998. TCimp= TCi + TCcb
8 Source: SCVWD Hydrology Procedures, 1998. TCperv= TCimp + TCp
9 Source: SCVWD Hydrology Procedures, 1998 (p.26, range); Nolte & Assoc 2000 uses 0.4 and 0.75.

Note: All Tc and R values less than 0.1 are rounded up to 0.1 for input into HEC-HMS.

Table 4-8: Time of Concentration and Storage Coefficient, Thompson Creek, Future Conditions, Rural and External Pervious Areas

	Subbasin	Length (ft)	Min. Elev. (ft)	Max. Elev. (ft)	Slope (ft/ft)	n		Tc (hr)	R / (Tc + R)	R
Rural	2	19200	310	2203	0.0986	0.070		0.701	0.750	2.103
Subbasins	3	10500	523	2449	0.1834	0.070		0.456	0.750	1.369
	4	10900	480	2403	0.1764	0.070		0.469	0.750	1.406
	7	3200	126	134	0.0025	0.070		0.716	0.750	2.149
	9	2300	125	134	0.0039	0.070		0.552	0.750	1.656
	10	9800	495	2418	0.1962	0.070		0.435	0.750	1.304
	11	16100	259	2384	0.1320	0.070		0.603	0.750	1.808
	12	10200	243	1842	0.1568	0.070		0.467	0.750	1.401
	21	14700	448	2273	0.1241	0.070		0.586	0.750	1.757
	22	17700	291	2269	0.1118	0.070		0.655	0.750	1.965
	26	11300	487	2341	0.1641	0.070		0.485	0.750	1.454
	27	12800	664	2153	0.1163	0.070		0.557	0.750	1.672
	29	9200	845	2480	0.1777	0.070		0.432	0.750	1.296
	32	8300	689	1789	0.1325	0.070		0.441	0.750	1.323
	33	9400	664	2246	0.1683	0.070		0.442	0.750	1.326
	34	10900	644	2491	0.1694	0.070		0.473	0.750	1.419
	38	6000	604	1417	0.1355	0.070		0.377	0.750	1.130
	40	4200	303	503	0.0476	0.070		0.407	0.750	1.222
	49	4300	552	1050	0.1158	0.070		0.334	0.750	1.002
							plus Urban Tc-Impervious			
External	6	1900	320	609	0.1521	0.070	0.129	0.343	0.750	1.028
Pervious	8	7700	246	1342	0.1423	0.070	0.129	0.548	0.750	1.643
Subbasins	13	3400	233	502	0.0791	0.070	0.152	0.479	0.750	1.438
	14	5000	279	854	0.1150	0.070	0.116	0.476	0.750	1.427
	15	3500	212	943	0.2089	0.070	0.108	0.372	0.750	1.117
	20	5200	506	920	0.0796	0.070	0.109	0.508	0.750	1.523
	24	4600	441	1182	0.1611	0.070	0.122	0.441	0.750	1.323
	37	3500	616	963	0.0991	0.070	0.121	0.435	0.750	1.306
	43	1300	520	823	0.2331	0.070	0.129	0.291	0.750	0.872

Notes:

Overland flow Manning's roughness coefficient 0.07 represents "sparse vegetation" (SCVWD, 1998).

The ratio of R / (Tc + R) lies between .5 and .9 for pervious and rural areas (SCVWD, 1998);

a ratio of 0.75 was chosen for rural hillside open space.

Table 4-9: Time of Concentration and Storage Coefficient, Ross Creek, Pre-Urban Conditions

Subbasin[1]	Tc (hr)	R / (Tc + R)	R
0	0.723	0.99	56
1	0.727	0.99	53
2	0.871	0.89	7
3	0.820	0.75	2.4
4	1.079	0.98	59
5	0.675	0.98	44
6	0.500	0.99	56
7	0.652	0.99	53
8	0.794	0.99	55
9	0.500	0.99	60
10	0.697	0.99	55
11	0.500	0.81	2.2
12	0.544	0.99	44
13	0.573	0.84	3
14	0.573	0.99	39
15	0.573	0.96	14.5
16	0.573	0.96	14.7
17	0.573	0.92	7

Notes:
1) All Pre-urban subbasins are considered rural

Table 4-10: Time of Concentration and Storage Coefficient, Ross Creek, Existing Conditions, Urban Areas

	Impervious			Pervious		
Subbasin	Tc (hr)	R / (Tc + R)	R	Tc (hr)	R / (Tc + R)	R
0	0.885	0.22	0.25	0.916	0.97	26.50
1	0.817	0.77	2.72	0.870	0.96	21.58
2	0.837	0.43	0.62	0.936	0.96	22.06
3	0.778	0.27	0.29	0.861	0.94	13.70
4	0.774	0.58	1.05	0.961	0.96	25.30
5	0.665	0.28	0.26	0.799	0.96	20.76
6	0.500	0.93	7.12	0.674	0.98	29.75
8	0.707	0.91	7.08	1.055	0.97	29.78
9	0.569	0.41	0.39	0.726	0.85	4.27
11	0.716	0.98	29.01	0.895	0.97	27.50
13	0.529	0.32	0.25	0.787	0.24	0.25

Table 4-11: Time of Concentration and Storage Coefficient, Ross Creek, Existing Conditions, Rural and External Pervious Areas

Subbasin	Type[1]	Tc (hr)	R / (Tc + R)	R
12	R	0.544	0.98	28.55
14	R	0.500	0.34	0.26
15	R	0.500	0.71	1.24
16	R	0.500	0.98	27.63
17	R	0.500	0.98	29.97
7	E	0.752	0.97	23.11
10	E	0.819	0.92	9.54

Notes:

1) R = Rural; E = Pervious urban external of storm drain system

Table 4-12: Time of Concentration and Storage Coefficient, Ross Creek, Future Conditions, Urban Areas

	Impervious			Pervious		
Subbasin	Tc (hr)	R / (Tc + R)	R	Tc (hr)	R / (Tc + R)	R
0	0.885	0.22	0.25	0.916	0.97	26.50
1	0.817	0.77	2.72	0.870	0.96	21.58
2	0.837	0.43	0.62	0.936	0.96	22.06
3	0.778	0.27	0.29	0.861	0.94	13.70
4	0.774	0.58	1.05	0.961	0.96	25.30
5	0.665	0.28	0.26	0.799	0.96	20.76
6	0.500	0.93	7.12	0.674	0.98	29.75
8	0.707	0.91	7.08	1.055	0.97	29.78
9	0.569	0.41	0.39	0.726	0.85	4.27
11	0.716	0.98	29.01	0.895	0.97	27.50
13	0.529	0.32	0.25	0.787	0.24	0.25

Table 4-13: Time of Concentration and Storage Coefficient, Ross Creek, Future Conditions, Rural and External Pervious Areas

Subbasin	Type[1]	Tc (hr)	R / (Tc + R)	R
12	R	0.544	0.98	28.55
14	R	0.500	0.34	0.26
15	R	0.500	0.71	1.24
16	R	0.500	0.98	27.63
17	R	0.500	0.98	29.97
7	E	0.752	0.97	23.11
10	E	0.819	0.92	9.54

Notes:

1) R = Rural; E = Pervious urban external of storm drain system

Table 4-12s: Time of Concentration and Storage Coefficient, Upper San Tomas Aquino Creek, Pre-Urban Condition

	Pervious		
Subbasin	Tc(hr)	R / (Tc + R)	R
PRE1	2.106	0.30	4.913
PRE2	1.674	0.30	3.905
PRE3a	0.500	0.30	1.170
PRE3b	0.500	0.30	1.170
PRE3c	0.723	0.30	1.687
PRE3d	1.221	0.30	2.849
PRE3e	0.590	0.30	1.378
PRE4	1.555	0.30	3.628
PRE5	1.701	0.30	3.970

Table 4-13s: Time of Concentration and Storage Coefficient, Upper San Tomas Aquino Creek, Existing Condition

	Pervious & Impervious		
Subbasin	Tc(hr)	R / (Tc + R)	R
EX1	0.476	0.60	0.317
EX2	0.538	0.50	0.538
EX3	0.448	0.60	0.298
EX4a	0.270	0.30	0.629
EX4b	0.418	0.30	0.974
EX4c	0.261	0.30	0.608
EX5	0.358	0.50	0.358
EX6	0.607	0.70	0.260
EX7	0.877	0.70	0.376
EX8	0.701	0.70	0.301
EX9	1.257	0.70	0.539
EX10	0.471	0.30	1.098
EX11a	0.500	0.30	1.170
EX11b	0.500	0.30	1.170

Table 4-14: Clark Unit Hydrograph Parameter Estimation: Storage Routing for Thompson Creek

STO i.d.	1		2		3		4		5		6		7		8		9		10		11	
area, ac (1)	46		69		26		313		38		159		184		532		250		73		920	
Levees? Y/N (2)	N		N		N		N		N		N		N		N		N		N		Y	
	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	0.2	3.3	0.3	5.0	0.1	1.9	1.2	22.5	0.1	2.8	0.6	11.4	0.7	13.2	2.1	23.3	1.0	18.0	0.3	5.2	3.6	40.3
	0.5	10.8	0.8	16.2	0.3	6.0	3.7	73.4	0.4	9.0	1.9	37.2	2.2	43.1	6.2	58.2	2.9	58.6	0.9	17.1	10.8	100.6
	0.7	12.9	1.1	19.5	0.4	7.3	4.9	88.1	0.6	10.8	2.5	44.7	2.9	51.7	8.3	66.5	3.9	70.3	1.1	20.5	14.4	115.0
	0.9	14.7	1.4	22.2	0.5	8.3	6.1	100.4	0.7	12.3	3.1	50.9	3.6	58.9	10.4	70.6	4.9	80.1	1.4	23.4	18.0	122.2
	1.4	17.2	2.2	25.9	0.8	9.7	9.8	117.5	1.2	14.4	5.0	59.6	5.7	69.0	16.6	103.9	7.8	93.7	2.3	27.3	28.8	126.5
	2.2	20.1	3.2	30.3	1.2	11.3	14.7	137.1	1.8	16.8	7.4	69.5	8.6	80.5	24.9	128.8	11.7	109.3	3.4	31.9	43.1	130.8
	12.9	20.1	19.5	30.3	7.3	11.3	88.1	137.1	10.8	16.8	44.7	69.5	51.7	80.5	149.6	128.8	70.3	109.3	20.5	31.9	258.8	130.8

12		13		14		15		16		17		18		19		20		21		22	
1202		999		1405		195		1308		517		465		1526		2543		1476		1559	
Y		Y		Y		Y		Y		Y		Y		Y		Y		Y		Y	
vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)
0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4.7	52.6	3.9	43.7	5.5	61.5	0.8	14.0	5.1	57.2	2.0	22.6	1.8	20.3	6.0	66.8	9.9	111.2	5.8	64.6	6.1	68.2
14.1	131.5	11.7	109.3	16.5	153.7	2.3	45.7	15.3	143.0	6.1	56.6	5.4	50.9	17.9	166.9	29.8	278.1	17.3	161.5	18.3	170.5
18.8	150.2	15.6	124.9	22.0	175.6	3.0	54.9	20.4	163.5	8.1	64.7	7.3	58.1	23.8	190.8	39.7	317.8	23.1	184.5	24.4	194.9
23.5	159.6	19.5	132.7	27.4	186.6	3.8	62.5	25.5	173.7	10.1	68.7	9.1	61.8	29.8	202.7	49.7	337.7	28.8	196.1	30.4	207.0
37.6	165.3	31.2	137.4	43.9	193.2	6.1	64.0	40.9	179.8	16.2	71.1	14.5	63.9	47.7	209.9	79.5	349.6	46.1	203.0	48.7	214.3
56.3	170.9	46.9	142.1	65.9	199.8	9.1	67.0	61.3	186.0	24.2	73.6	21.8	66.1	71.5	217.0	119.2	361.5	69.2	209.9	73.1	221.6
338.0	170.9	281.1	142.1	395.2	199.8	54.9	67.0	367.8	186.0	145.5	73.6	130.8	66.1	429.2	217.0	715.1	361.5	415.2	209.9	438.4	221.6

Unitized Storage-Discharge Relationships (3)

condition								
1 A > 400 ac w/o levees								
vol (ac-ft/sq mi)	0	2.5	7.5	10	12.5	20	30	180
Q (cfs/sq mi)	0	28	70	80	85	125	155	155
2 A > 400 ac w/ levees								
vol (ac-ft/sq mi)	0	2.5	7.5	10	12.5	20	30	180
Q (cfs/sq mi)	0	28	70	80	85	88	91	91
3 A < 400 ac w/o levees								
vol (ac-ft/sq mi)	0	2.5	7.5	10	12.5	20	30	180
Q (cfs/sq mi)	0	46	150	180	205	240	280	280
4 A < 400 ac w/ levees								
vol (ac-ft/sq mi)	0	2.5	7.5	10	12.5	20	30	180
Q (cfs/sq mi)	0	46	150	180	205	210	220	220

E = Pervious "External"
I = Impervious
P = Pervious

(1) "Total area" reflects sum of Pervious, Impervious and Undeveloped areas contributing to storm drains for each STO module

(2) Areas upstream of Yerba Buena were assigned "w/o levee" status, downstream of Yerba Buena assigned "w/levee" status following general guidance in SCVWD, 1998

(3) Tables derived from plot in SCVWD, 1998 Fig 7

Table 4-15: HEC-HMS Reach Routing, Thompson Creek, Pre-Urban Conditions

HMS ID	Creek	Connection	Shape	Length, ft	Slope, ft/ft	Width or Diameter, ft	H:V	Manning's n	Source Information (width, diameter, H:V, & n)
1	Thompson	27TR - J-1	Prism	375	0.02670	4.00	0.80	0.040	A
2	Thompson	J-1 - J-2	Prism	2188	0.02513	7.00	0.40	0.045	B
3	Hawk	33HR - J-2	Prism	1166	0.04290	4.00	0.80	0.040	A
4	Thompson	J-2 - J-3	Prism	2559	0.01563	7.00	0.40	0.045	B
5	Misery	34MR - J-3	Prism	2237	0.03800	4.00	0.80	0.040	A
6	Thompson	J-3 - J-5	Prism	5420	0.01753	7.00	0.40	0.045	B
7		38CR - J-4	Prism	2600	0.02671	4.00	0.80	0.035	A, B
8	Cribari	29CR - J-4	Prism	7400	0.04912	6.00	0.80	0.035	A, B
9	Cribari	J-4 - J-5	Prism	1640	0.03173	7.00	0.80	0.035	A, B
10	Thompson	J-5 - J-6	Prism	3300	0.01667	7.00	1.00	0.050	B
11	Thompson	J-6 - J-7	Prism	6030	0.01658	9.00	0.10	0.050	B
12	Yerba Buena	21YR - J-7	Prism	4835	0.02792	11.00	2.10	0.030	B
13	Thompson	J-7 - J-9	Prism	2871	0.01045	9.00	1.00	0.040	A, B
14	Evergreen	J-8 - J-9	Prism	414	0.03000	8.00	2.00	0.030	A, B
16	Thompson	J9 - J-10	Prism	6366	0.00943	12.00	1.04	0.035	B
17		26WR - J-10	Prism	9700	0.03062	10.00	0.80	0.035	A, B
18	Fowler	3WR - J-10	Prism	9700	0.03062	10.00	0.80	0.035	A, B
19	Thompson	J-10 - J-11	Prism	3552	0.01056	13.00	1.64	0.045	B
20	Quimby	4QR - J-11	Prism	9700	0.03267	10.00	0.80	0.035	A, B
21	Thompson	J-11 - J-12	Prism	5837	0.00728	14.00	0.61	0.030	B
22	Norwood	10NR - J-12	Prism	14000	0.02843	10.00	0.80	0.035	A, B
23	Thompson	J-12 - J-13	Prism	2634	0.00627	20.48	1.00	0.031	B, D
24	Thompson	J-13 - J-14	Prism	4217	0.00363	37.33	1.00	0.033	B, D
25	Flint	11FR - J-14	Prism	6400	0.01927	10.00	0.80	0.035	A, B
26	Lower Silver	J-14 - J-15	Prism	4379	0.00200	47.70	2.00	0.035	D
27	South Babb	22BR - J-15	Prism	8454	0.01881	15.00	1.50	0.030	A, B
28	Lower Silver	J-15 - J-16	Prism	3235	0.00200	37.00	1.23	0.035	D
29	North Babb	STO-19 - J-16	Prism	1307	0.00612	10.00	1.00	0.025	A, B
30	Lower Silver	J-16 - J-17	Prism	12041	0.00200	41.00	0.75	0.035	D
31	Lower Silver	J-17 - J-18	Prism	1100	0.00200	46.00	0.01	0.035	D
32	Miguelita	12GR - J-18	Prism	14200	0.00853	10.00	0.80	0.035	A, B, D
33	Lower Silver	J-18 - J-19	Prism	2020	0.00200	65.00	0.96	0.035	D

Lengths approximated by querying WLA creek and storm drain GIS layers.
Slopes approximated using length estimates and USGS DEM elevations.

Past reach lengths validated or changed based on 1964 arial photographs (Pafford & Associates, suveyors).
Existing storm drains throughout watershed NOT present in 1964; historic channel charactersitcs estimated for these reaches.

Source information used to approximate width, diameter, H:V, and Manning's n:
A - Nolte (2000)
B - PWA and Balance Hydrologics survey (2002)
D - Lower Silver HEC-RAS model

Note: there is no R-15

Table 4-16: HEC-HMS Reach Routing, Thompson Creek, Existing and Future Conditions

HMS ID	Creek	Connection	Shape	Length, ft	Slope, ft/ft	Width or Diameter, ft	H:V	Manning's n	Source Information (width, diameter, H:V, & n)
1	Thompson	27TR - J-1	Prism	375	0.02670	4.00	0.80	0.040	A
2	Thompson	J-1 - J-2	Prism	2188	0.02513	7.00	0.40	0.045	B
3	Hawk	33HR - J-2	Prism	1166	0.04290	4.00	0.80	0.040	A
4	Thompson	J-2 - J-3	Prism	2559	0.01563	7.00	0.40	0.045	B
5	Misery	34MR - J-3	Prism	2237	0.03800	4.00	0.80	0.040	A
6	Thompson	J-3 - J-5	Prism	5420	0.01753	7.00	0.40	0.045	B
7		38CR - J-4	Circle	2991	0.02671	3.02		0.013	C
8	Cribari	29CR - J-4	Circle	6658	0.04912	4.38		0.014	C
9	Cribari	J-4 - J-5	Circle	1640	0.03173	5.50		0.013	C
10	Thompson	J-5 - J-6	Prism	3300	0.01667	7.00	1.00	0.050	B
11	Thompson	J-6 - J-7	Prism	6030	0.01658	9.00	0.10	0.050	B
12	Yerba Buena	21YR - J-7	Prism	4835	0.02792	11.00	2.10	0.030	B
13	Thompson	J-7 - J-9	Prism	2871	0.01045	9.00	1.00	0.040	A, B
14	Evergreen	J-8 - J-9	Prism	414	0.03000	8.00	2.00	0.030	A, B
16	Thompson	J9 - J-10	Prism	6366	0.00943	12.00	1.04	0.035	B
17		26WR - J-10	Circle	9178	0.03062	6.65		0.013	C
18	Fowler	3WR - J-10	Circle	9178	0.03062	6.65		0.013	C
19	Thompson	J-10 - J-11	Prism	3552	0.01056	13.00	1.64	0.045	B
20	Quimby	4QR - J-11	Circle	9762	0.03267	6.13		0.013	C
21	Thompson	J-11 - J-12	Prism	5837	0.00728	14.00	0.61	0.030	B
22	Norwood	10NR - J-12	Circle	12371	0.02843	6.34		0.013	C
23	Thompson	J-12 - J-13	Prism	2634	0.00627	20.48	1.00	0.031	B, D
24	Thompson	J-13 - J-14	Prism	4217	0.00363	37.33	1.00	0.033	B, D
25	Flint	11FR - J-14	Circle	6538	0.01927	6.87		0.013	C
26	Lower Silver	J-14 - J-15	Prism	4379	0.00200	47.70	2.00	0.035	D
27	South Babb	22BR - J-15	Prism	8454	0.01881	15.00	1.50	0.030	A, B
28	Lower Silver	J-15 - J-16	Prism	3235	0.00200	37.00	1.23	0.035	D
29	North Babb	STO-19 - J-16	Prism	1307	0.00612	10.00	1.00	0.025	A, B
30	Lower Silver	J-16 - J-17	Prism	12041	0.00200	41.00	0.75	0.035	D
31	Lower Silver	J-17 - J-18	Prism	1100	0.00200	46.00	0.01	0.035	D
32	Miguelita	12GR - J-18	Circle	15570	0.00853	6.50		0.013	C
33	Lower Silver	J-18 - J-19	Prism	2020	0.00200	65.00	0.96	0.035	D

Lengths approximated by querying WLA creek and storm drain GIS layers.
Slopes approximated using length estimates and USGS DEM elevations.

Source information used to approximate width, diameter, H:V, and Manning's n:

A - Nolte (2000)
B - PWA and Balance Hydrologics survey (2002)
C - Storm drain analysis
D - Lower Silver HEC-RAS model

Note: there is no R-15

Table 4-17: HEC-HMS Reach Routing, Ross Creek, Pre-Urban Conditions

HMS ID	Connection	Shape	Length, ft	Slope, ft/ft	Width, ft	H:V	Manning's n
0	J0 - J1	Prism	2600	0.0027	12.0	1.4	0.040
1	J1 - J2	Prism	5200	0.0038	12.0	1.4	0.040
2	J2 - J3	Prism	4200	0.0060	12.0	1.4	0.045
3	J3 - SB6	Prism	6300	0.0071	10.0	1.1	0.060
4	J3 - J4	Prism	6100	0.0082	12.0	1.1	0.050
5	J4 - J5	Prism	7900	0.0089	12.0	1.1	0.060
6	J5 - J6	Prism	900	0.0111	12.0	1.1	0.070
7	J6 - SB14	Prism	5100	0.0108	6.0	1.6	0.055
8	J6b - J7	Prism	5600	0.0254	6.0	1.6	0.055
8b	J6 - J6b	Prism	100	0.0254	6.0	1.6	0.055
9	J7 - SB17	Prism	1400	0.0571	3.0	2.2	0.045

Table 4-18: HEC-HMS Reach Routing, Ross Creek, Existing and Future Conditions

HMS ID	Connection	Shape	Length, ft	Slope, ft/ft	Width, ft	H:V	Manning's n
0	J0 - J1	Prism	2600	0.0027	12.0	1.4	0.040
1	J1 - J2	Prism	5200	0.0038	12.0	1.4	0.040
2	J2 - J3	Prism	4200	0.0060	12.0	1.4	0.045
3	J3 - SB6	Prism	6300	0.0071	10.0	1.1	0.060
4	J3 - J4	Prism	6100	0.0082	12.0	1.1	0.050
5	J4 - J5	Prism	7900	0.0089	12.0	1.1	0.060
6	J5 - J6	Prism	900	0.0111	12.0	1.1	0.070
7	J6 - SB14	Prism	5100	0.0108	6.0	1.6	0.055
8	J6b - J7	Prism	5700	0.0254	6.0	1.6	0.055
8b	J6 - J6b	Prism	100	0.0254	6.0	1.6	0.055
9	J7 - SB17	Prism	1400	0.0571	3.0	2.2	0.045

Table 4-17s: HEC-HMS Reach Routing, Upper San Tomas Aquino Creek, Pre-Urban Condition

HMS ID	Connection	Shape	Length, ft	Slope, ft/ft	Width, ft	H:V	Manning's N
R1	J3e-J3	PRISM	6300	0.0056	13	1	0.035
R2	J3-J5	PRISM	9250	0.0041	15	1.5	0.035
R3	J5-J6	PRISM	4150	0.0046	15	1.5	0.035
R3b	PRE3a-J3b	PRISM	4640	0.0216	8	1	0.035
R3c	J3b-J3c	PRISM	5000	0.0206	8	1	0.035
R3d	J3c-J3d	PRISM	5260	0.0165	12	2	0.035
R3e	J3d-J3e	PRISM	3665	0.0147	12	2	0.035

Table 4-18s: HEC-HMS Reach Routing, Upper San Tomas Aquino Creek, Existing Condition

HMS ID	Connection	Shape	Length, ft	Slope, ft/ft	Width, ft	H:V	Manning's N
R1	EX1-J1	PRISM	4200	0.0009	6	1	0.035
R2	J1-J3	PRISM	1400	0.0009	6	1	0.035
R4	EX10-J2	PRISM	19100	0.0139	8	0.8	0.035
R4a	J11a-J4b	PRISM	5260	0.0238	8	1	0.035
R4b	J4a-J4b	PRISM	5000	0.014	8	1	0.015
R4c	J4c-J1	PRISM	4640	0.0108	12	1	0.012
R5	J2-J1	PRISM	300	0.0139	8	0.8	0.035
R6	J3-J5	PRISM	6300	0.0056	13	1	0.012
R7	EX2-J4	PRISM	9400	0.0074	7	0.1	0.012
R8	J4-J5	PRISM	300	0.0074	7	0.1	0.012
R9	J5-J6	PRISM	13560	0.0044	15	1.5	0.012
R11b	J11a-J11b	PRISM	3660	0.0273	8	1	0.035

Table 4-19: Soil Moisture Accounting Parameter Initialization

Input Parameter	Source	Method
Maximum Canopy Interception Storage	Land use layer (GIS) from William Lettis & Assoc. (WLA) Vegetation-interception relationships from literature Mean Annual Precipitation (MAP) data from SCVWD	1. Interception storage assigned for each vegetation type present using published data from similar environments (% of MAP) 2. MAP assigned for each drainage basin 3. Interception converted to depth using precipitation depth for frequent design storm from SCVWD 1998 (1"); depths verified vs. values from literature
Maximum Surface Depression Storage	Land use data (GIS) from WLA HEC-HMS Manual	1. Surface storage depression value assigned per land use (GIS) using HEC-HMS manual guidance 2. Drainage area surface storage determined as a weighted average of land use using GIS data
Maximum Infiltration Rate	Soil data from SCVWD database Soil parameters estimated from the literature (Rawls et al., 1982)	1. The initial maximum infiltration rate was estimated for each of the Hydrologic Soil Groups present (B, C, and D) using the Green-Ampt equation and soil parameter estimates from the literature. 2. All infiltration rates were then scaled up during model verification
Maximum Percolation Rate	Soil data from SCVWD database Saturated hydraulic conductivity (K_s) for representative soils estimated from literature	1. The Maximum Percolation Rate was set equal to saturated hydraulic conductivity (K_s)
Maximum Soil Profile Storage	Soil data from SCVWD database Soil profile depth from Santa Clara County Soil Survey (NRCS) and effective porosity from Rawls et al, 1982	1. Soil profile storage for each soil type taken from SCVWD data (in GIS) as a product of effective soil depth (root zone depth) and effective porosity. 2. Weighted average Maximum Soil Profile Storage determined for each drainage area using SCVWD data in GIS
Tension (Unsaturated) Zone Storage	Soil data from SCVWD database HEC-HMS manual and Santa Clara County Soil Survey (NRCS)	1. Tension zone storage estimated as Available Water holding Capacity (A.W.C) from the County Soil Survey
Maximum Groundwater Layer Storage	N/A	1. Following guidance from the literature, groundwater storage and percolation rate were not considered in this assessment due to the lack of base flow in the creeks

Table 4-20: Simulated and Measured Thompson Creek Discharge Volumes and Precipitation, Sping 2003

	Location	Thompson Creek at Quimby Road		Yerba Buena Creek watershed	
Event	HEC-HMS location Gage name	J-11 TCQR gage	% of precip volume	STO-8 + R-12 YBSF gage	% of precip volume
	Drainage area, sq-mi	17.37		2.58	
	Drainage area, ac	11115		1650	
Gage time period		Jan 3 (12:00) to Apr 9 (24:00)		Jan 8 (17:00) to Mar 18 (14:00)	
	Precipitation, in	3.685		3.024	
	Precipitation, ac-ft	3412.9		415.8	
	HEC Continuous., ac-ft	168.8	4.9	14.7	3.5
	Gage, ac-ft	163.3	4.8	2.4	0.6
January 9-10	Precipitation, in	0.564		0.564	
	Precipitation, ac-ft	522.4		77.5	
	HEC Event-based, ac-ft	51.7	9.9	5.0	6.5
	HEC Continuous, ac-ft	35.2	6.7	3.1	3.9
	Gage, ac-ft	33.6	6.4	1.1	1.4
February 16	Precipitation, in	0.420		0.420	
	Precipitation, ac-ft	388.8		57.7	
	HEC Event-based, ac-ft	32.9	8.5	3.1	5.4
	HEC Continuous, ac-ft	14.6	3.8	1.1	1.9
	Gage, ac-ft	19.8	5.1	0.2	0.3
February 25-27	Precipitation, in	0.950		0.950	
	Precipitation, ac-ft	879.7		130.6	
	HEC Event-based, ac-ft	119.7	13.6	12.9	9.9
	HEC Continuous, ac-ft	76.2	8.7	6.9	5.3
	Gage, ac-ft	40.7	4.6	0.4	0.3
March 15-16	Precipitation, in	0.657		0.657	
	Precipitation, ac-ft	608.9		90.4	
	HEC Event-based, ac-ft	64.1	10.5	6.4	7.1
	HEC Continuous, ac-ft	41.5	6.8	3.6	4.0
	Gage, ac-ft	39.2	6.4	0.5	0.5

Table 4-21: Thompson Creek, Event-Based Design Storm Results

	Q-peak, cfs						V-total, ac-ft					
Return Period (years)	**2**	**5**	**10**	**25**	**50**	**100**	**2**	**5**	**10**	**25**	**50**	**100**
Pre-Urban SCS (24-hr balanced)												
Junction J-3	127.2	311.4	454.7	643.0	781.7	920.1	63.6	155.0	226.7	324.6	400.2	477.2
J-5	189.1	451.4	653.4	917.7	1112.6	1305.1	99.2	235.8	342.2	486.5	597.8	710.7
J-7	274.2	650.7	937.3	1313.7	1592.1	1868.8	154.1	365.9	530.8	754.2	926.6	1101.5
J-9	321.5	763.6	1098.8	1539.1	1866.4	2188.7	181.4	432.2	627.5	892.6	1097.2	1304.7
J-10	438.1	1032.6	1479.7	2064.1	2496.4	2922.3	252.2	595.2	861.5	1221.8	1499.5	1781.0
J-11	529.6	1229.4	1751.6	2430.6	2933.6	3428.3	316.6	735.9	1059.5	1496.0	1831.7	2171.6
J-12	660.4	1494.2	2110.2	2909.3	3499.4	4081.0	434.0	980.2	1397.3	1957.0	2385.7	2818.9
Subbasin SB-27	55.7	138.7	203.1	289.0	353.0	416.2	28.0	69.7	102.6	147.6	182.5	218.0
SB-21	57.2	137.6	198.8	279.7	340.3	400.0	29.2	70.9	103.5	147.8	182.2	217.0
SB-41	13.7	32.6	46.7	65.0	78.7	92.3	12.2	29.9	43.8	62.6	77.2	92.0
Reach-12	57.2	137.6	198.7	279.6	340.0	399.7	29.2	70.9	103.5	147.8	182.2	217.0
Total Yuerba Buena	69.2	166.4	239.1	336.5	409.8	482.8	41.5	100.8	147.3	210.5	259.3	309.0
SB-39	10.4	19.3	25.4	33.0	38.5	43.8	8.2	16.2	21.9	29.3	34.8	40.3
Exsiting SCS (24-hr balanced)												
Junction J-3	136.4	335.6	486.0	679.0	820.6	960.3	66.8	158.8	230.9	329.0	404.8	481.9
J-5	266.6	591.9	812.3	1082.9	1278.9	1473.4	129.2	276.2	388.6	539.7	655.6	772.9
J-7	442.3	918.2	1244.0	1630.4	1907.2	2168.0	222.8	456.4	633.2	869.7	1050.6	1233.1
J-9	495.5	1045.4	1423.2	1876.4	2199.0	2510.1	252.8	525.1	731.9	1009.1	1221.4	1435.8
J-10	712.9	1389.6	1856.2	2420.0	2825.7	3215.9	378.2	754.8	1038.2	1416.2	1704.8	1995.9
J-11	924.1	1671.3	2182.7	2798.4	3244.6	3670.0	498.9	965.0	1312.2	1772.8	2123.3	2476.0
J-12	1306.6	2129.7	2683.7	3351.7	3836.4	4298.3	756.7	1383.1	1840.7	2441.6	2895.9	3351.4
Subbasin SB-27	55.7	138.7	203.1	289.0	353.0	416.2	28.0	69.7	102.6	147.6	182.5	218.0
SB-20	22.8	42.1	55.1	70.7	81.8	92.6	10.0	19.2	25.8	34.3	40.6	46.9
SB-21	57.2	137.6	198.8	279.7	340.3	400.0	29.2	70.9	103.5	147.8	182.2	217.0
SB-41-Impervious	140.5	196.3	231.7	274.4	304.5	334.5	22.9	34.1	41.4	50.3	56.8	63.1
SB-41-Pervious	6.4	28.2	45.4	68.0	85.1	102.3	2.3	6.8	10.5	15.7	19.8	24.1
Reach-12	57.2	137.6	198.7	279.6	340.0	399.7	29.2	70.9	103.5	147.8	182.2	217.0
STO-8	60.0	83.9	104.3	121.8	128.8	128.8	35.3	60.1	77.7	100.3	117.2	134.0
Total Yuerba Buena	115.8	221.3	301.3	396.5	467.9	528.5	64.5	131.0	181.2	248.2	299.4	351.0
SB-39-Impervious	70.4	98.4	116.1	137.5	152.5	167.5	11.5	17.1	20.8	25.3	28.5	31.7
SB-39-Pervious	14.6	31.4	43.1	57.5	68.0	78.3	3.5	7.5	10.4	14.3	17.3	20.3
Total SB 39	82.9	126.0	154.1	188.4	212.8	237.0	15.0	24.6	31.2	39.6	45.8	51.9
Future SCS (24-hr balanced)												
Junction J-3	181.7	404.5	562.1	761.5	907.5	1050.0	86.5	189.7	268.1	373.2	453.5	534.6
J-5	356.3	699.9	921.9	1199.6	1401.4	1595.9	165.7	329.4	451.1	612.3	734.6	857.5
J-7	586.0	1098.6	1418.8	1810.1	2082.1	2342.6	293.1	555.7	748.3	1001.7	1193.2	1385.1
J-9	696.7	1297.8	1676.1	2137.2	2459.4	2769.9	350.7	664.2	893.6	1194.8	1422.4	1650.1
J-10	962.9	1706.8	2179.3	2757.2	3164.2	3555.2	504.0	935.2	1248.5	1658.4	1967.3	2276.1
J-11	1187.7	2001.9	2519.3	3152.8	3595.5	4025.6	644.7	1172.9	1554.0	2050.7	2424.0	2796.8
J-12	1584.3	2476.2	3036.9	3724.3	4204.8	4673.7	922.6	1618.2	2113.4	2754.5	3234.0	3711.6
Subbasin SB-27	71.5	163.3	232.2	321.9	387.5	452.3	35.1	81.3	116.8	164.8	201.5	238.8
SB-20	47.6	67.5	79.8	94.2	104.3	114.0	22.0	34.1	42.0	51.8	58.9	65.8
SB-21	79.1	170.3	236.8	322.5	385.1	446.1	39.3	87.0	123.1	171.2	208.0	244.9
SB-41-Impervious	161.1	225.2	265.8	314.8	349.2	383.6	26.4	39.3	47.6	57.9	65.4	72.6
SB-41-Pervious	5.7	25.0	40.1	60.0	74.9	90.0	2.0	5.7	8.9	13.3	16.8	20.3
Reach-12	79.0	170.3	236.8	322.2	384.8	445.8	39.4	87.0	123.1	171.2	208.0	245.0
STO-8	72.6	104.7	118.7	128.8	128.8	128.8	50.3	79.1	98.5	123.0	141.0	158.8
Total Yuerba Buena	151.6	274.1	352.0	451.0	513.6	574.6	89.7	166.1	221.5	294.2	349.0	403.7
SB-39-Impervious	94.9	132.6	156.5	185.3	205.6	225.8	15.7	23.3	28.3	34.4	38.8	43.1
SB-39-Pervious	10.9	23.2	31.7	42.3	50.0	57.5	2.4	5.0	7.0	9.6	11.6	13.6
Total SB-39	104.6	153.7	185.5	224.1	251.5	278.8	18.0	28.3	35.3	44.0	50.4	56.7

	2	**5**	**10**	**20**	**50**	**100**
Historical Flood Quantile (SCVWD, 1998)						
Junction J-3	171.9	310.6	426.9	553.5	750.3	923.0
J-5	252.1	460.4	639.0	837.0	1149.7	1429.2
J-7	339.7	640.9	907.6	1209.8	1697.3	2142.5
J-12	903.1	1583.0	2172.1	2829.0	3880.3	4838.1

Table 4-22: Pre-Urban Gage 21 Calibration - Period from 10/1/1950 - 3/31/1952

	Volume (ac-ft)	Average Q (cfs)	Model Results' RMS Error	Percent Error in Volume
Gage 21	2,500	2.3	30	12%
Gage 21 - Flows < 1 cfs	2,350	2.2	31	7%
Model Results @ J-5	2,190	2.0	-	-

Table 4-23: Pre-Urban Gage 51 Calibration - Period from 10/1/1956 - 5/1/1958

	Volume (ac-ft)	Average Q (cfs)	Model Results' RMS Error	Percent Error in Volume
Gage 51	3,260	2.9	95	34%
Gage 51 - Flows < 1 cfs	2,130	2.0	46	-1%
Model Results @ J-1	2,160	1.9	-	-

Table 4-24: Existing Condition Gage 21 Calibration - Period from 11/1/2000 - 3/31/2003

	Volume (ac-ft)	Average Q (cfs)	Model Results' RMS Error	Percent Error in Volume
Gage 21	2,810	3.0	44	61%
Gage 21 - Flows < 1 cfs	2,510	2.6	47	56%
Model Results @ J-5	1,100	1.2	-	-

Table 4-25: Existing Condition Gage 51 Calibration - Period from 11/1/200 - 3/31/2003

	Volume (ac-ft)	Average Q (cfs)	Model Results' RMS Error	Percent Error in Volume
Gage 51	5,080	5.0	126	3%
Gage 51 - Flows < 1 cfs	4,730	4.6	130	-4%
Model Results @ J-1	4,910	4.9	-	-

Figures

FIGURES

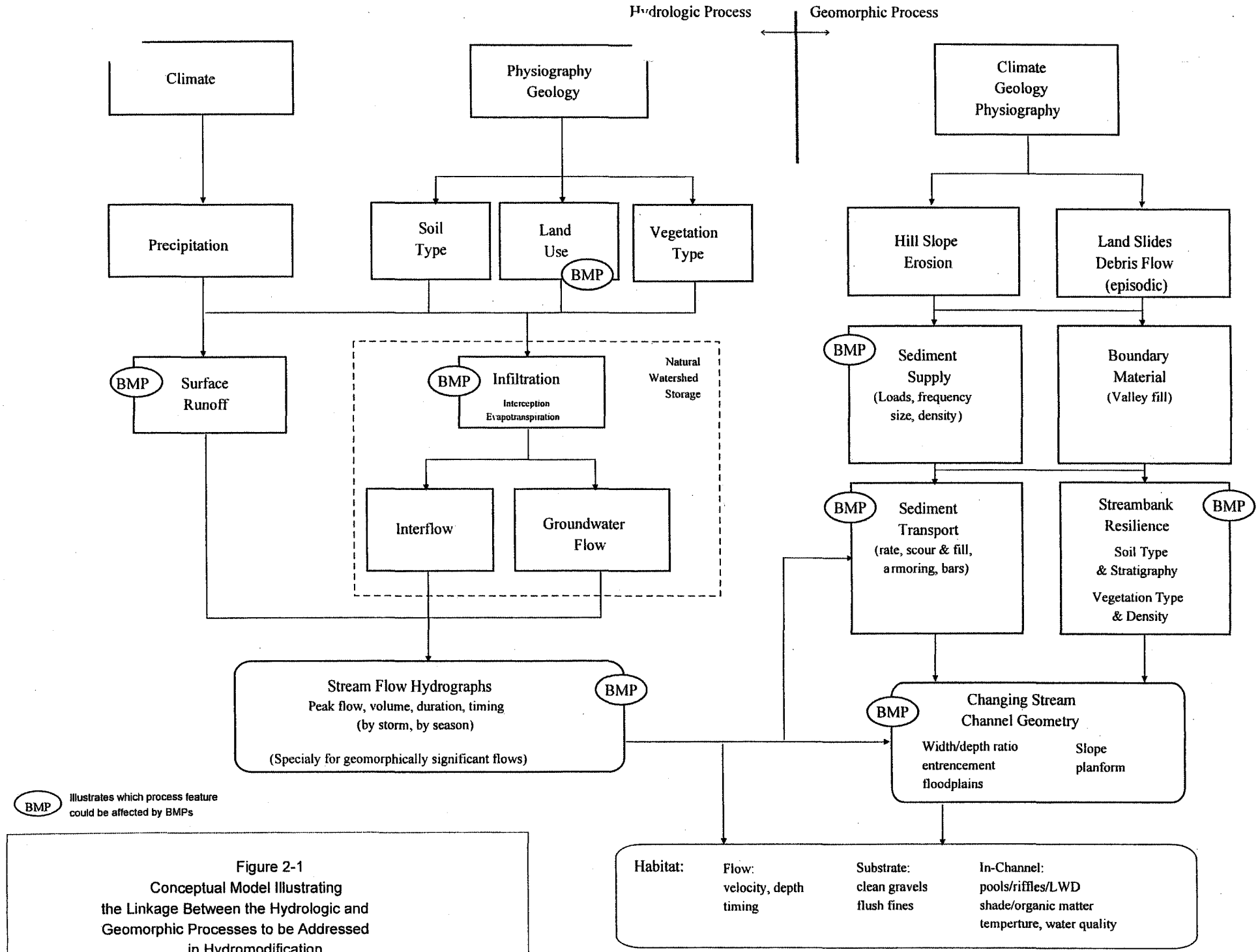
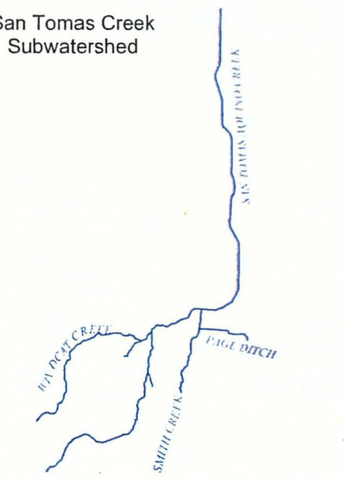
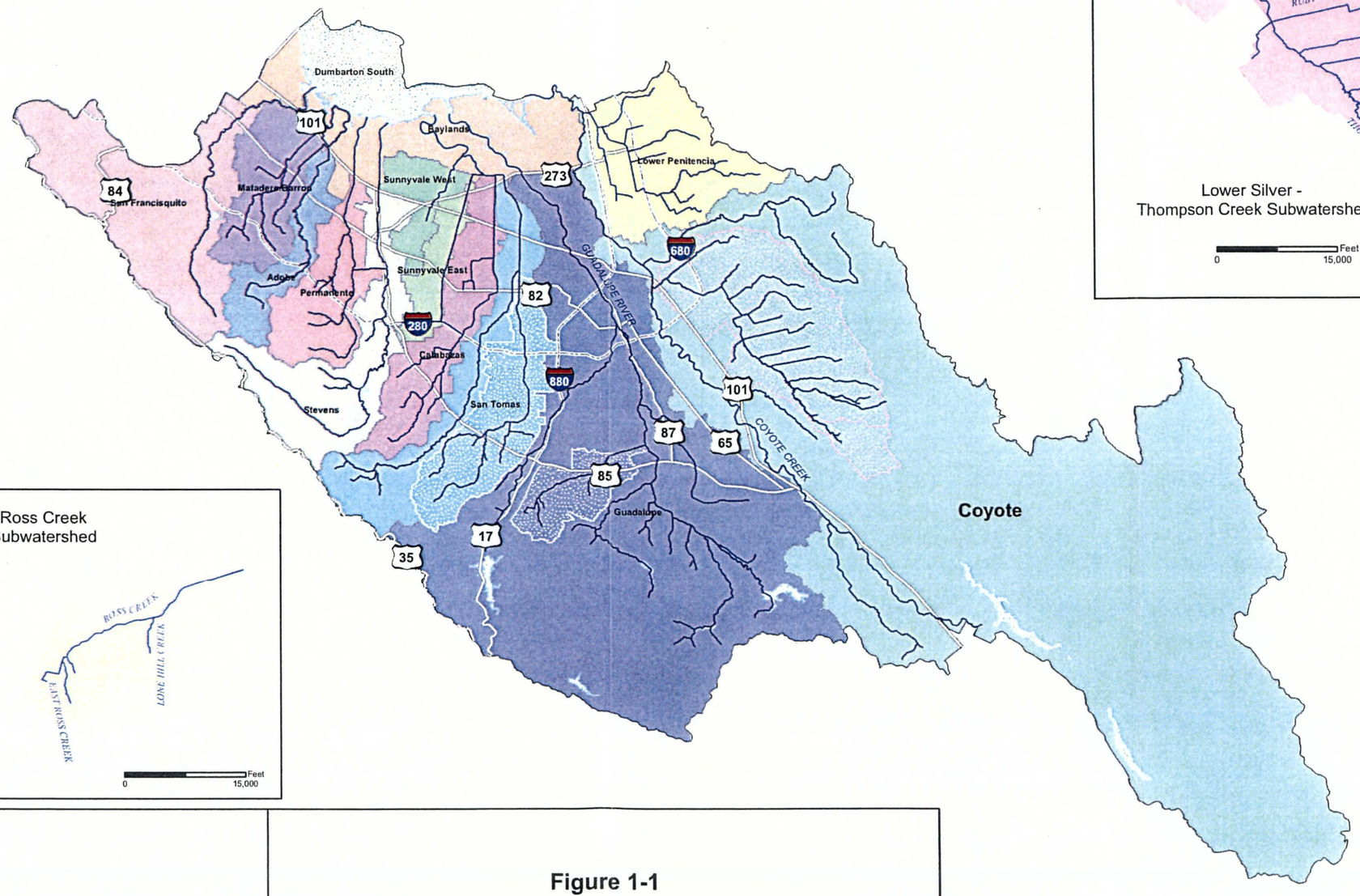


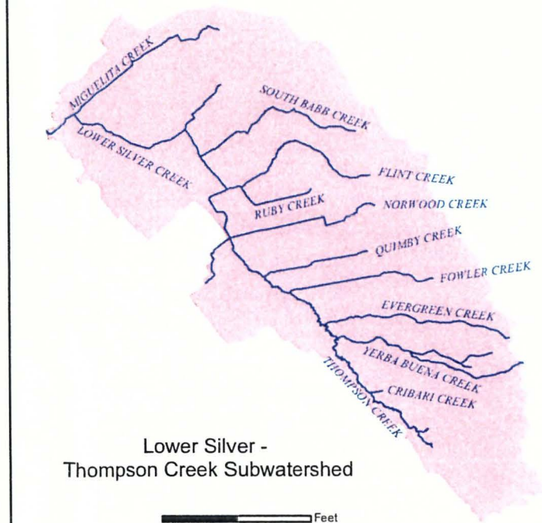
Figure 2-1
Conceptual Model Illustrating
the Linkage Between the Hydrologic and
Geomorphic Processes to be Addressed
in Hydromodification

San Tomas Creek
Subwatershed





Lower Silver -
Thompson Creek Subwatershed



Ross Creek
Subwatershed

Legend

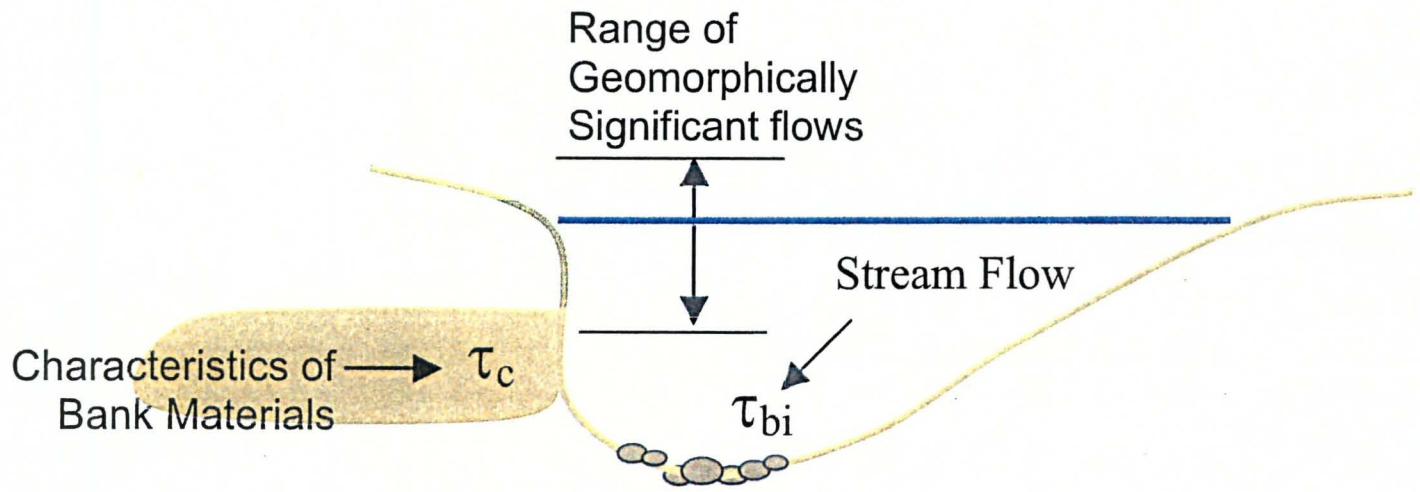
- Major Creeks
- Roadway
- Interstate
- Highway

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Figure 1-1
Santa Clara Basin and Test Watersheds

Feet
0
50,000

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$$W = \sum_{i=1}^{n} \left(\tau_{bi} - \tau_c\right)^{1.5} \cdot V \cdot \Delta t$$

Figure 2-2
Schematic Illustrating Effective Work Index

SCVWD Hydromodification Management Plan
Draft Report

May 2004

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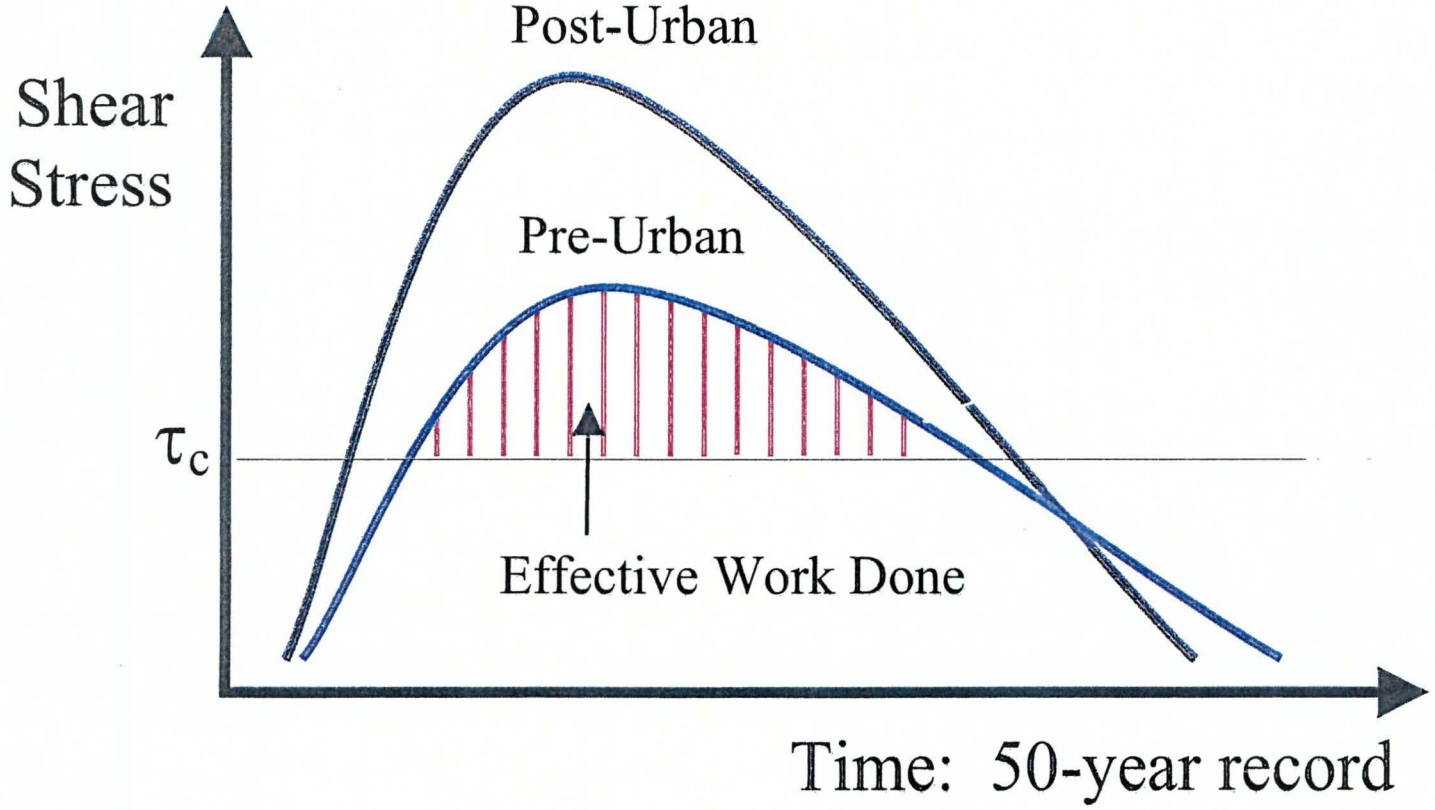


Figure 2-3
Definition of Ep Ratio

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Figure 3-1
Cross-section survey at YB1-4. A tape was stretched between two pins to measure the channel dimensions,bed material, bank material, vegetation, roughness and other characteristics were described.



Cross Section Location

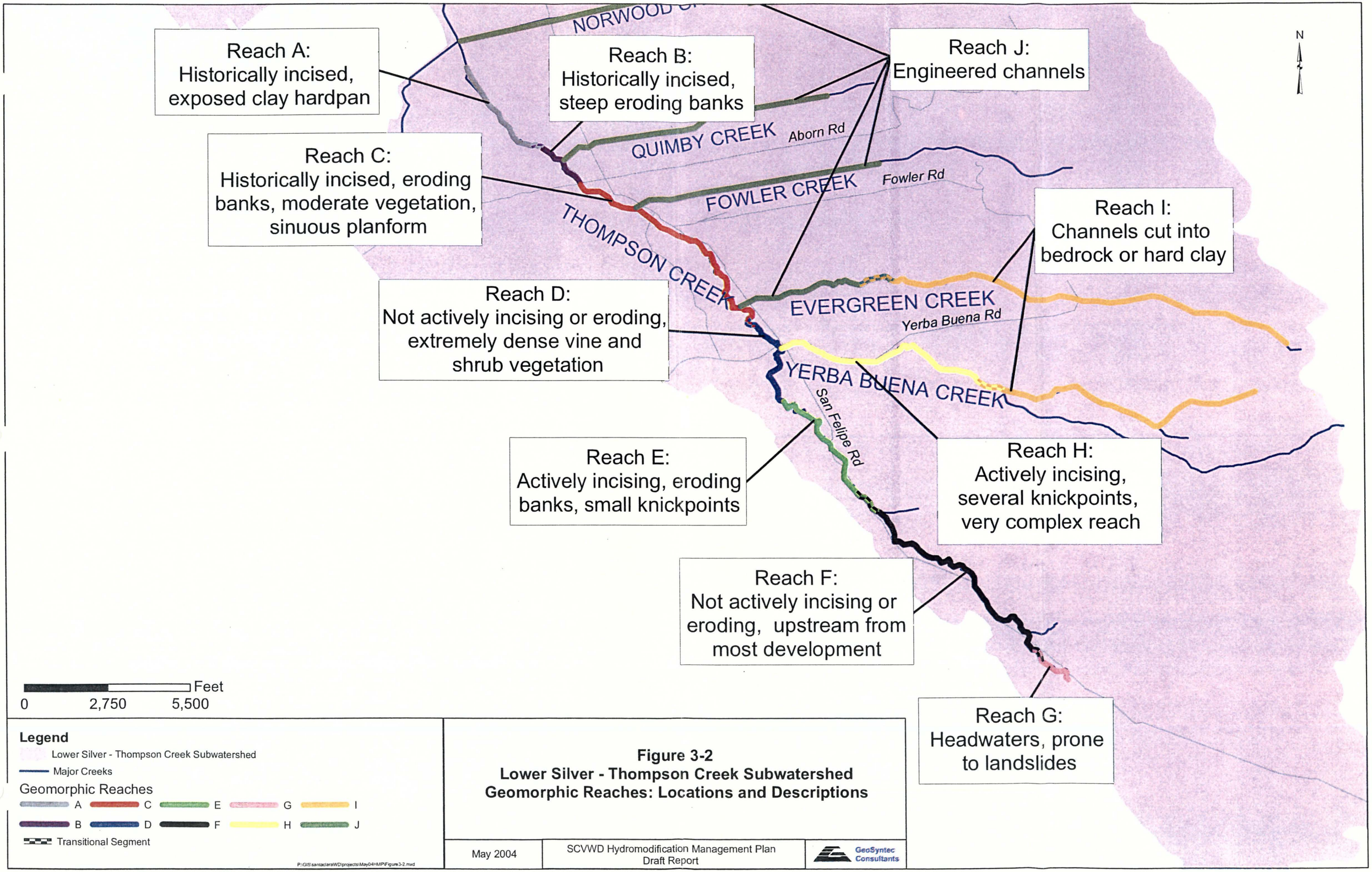
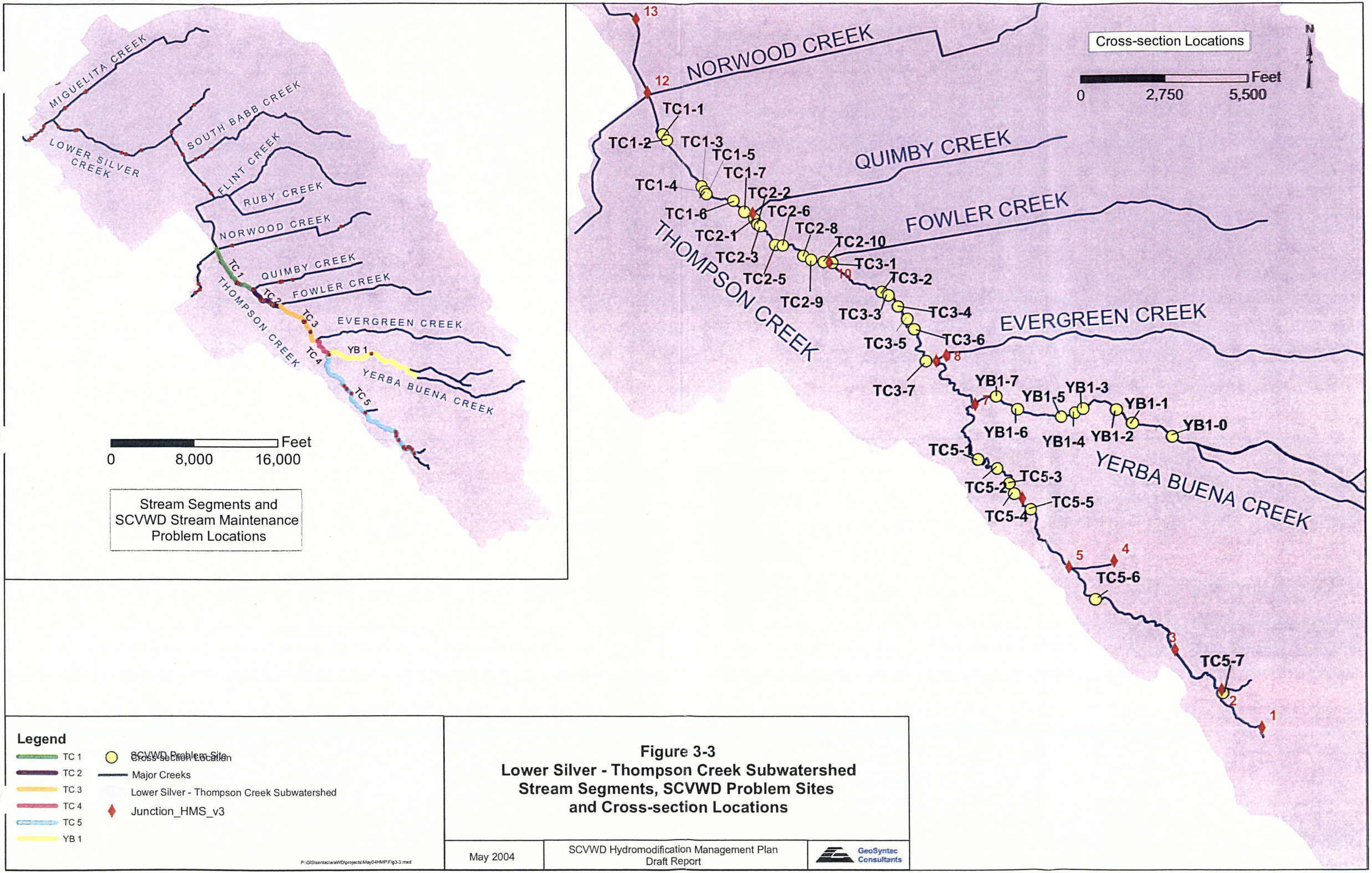


Figure 3-2
Lower Silver - Thompson Creek Subwatershed
Geomorphic Reaches: Locations and Descriptions



Legend

- TC 1
- TC 2
- TC 3
- TC 4
- TC 5
- YB 1
- SCVWD Problem Site
- Cross-section Location
- Major Creeks
- Lower Silver - Thompson Creek Subwatershed
- Junction_HMS_v3

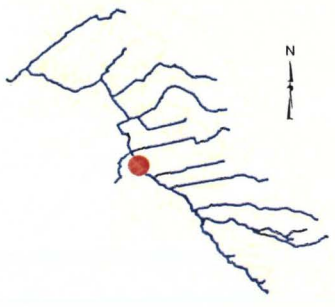
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Figure 3-3
Lower Silver - Thompson Creek Subwatershed
Stream Segments, SCVWD Problem Sites and Cross-section Locations

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Figure 3-4
Photo showing a large grade control structure in Segment TC1. Note the exposed clay-silt hardpan just down stream from the structure.



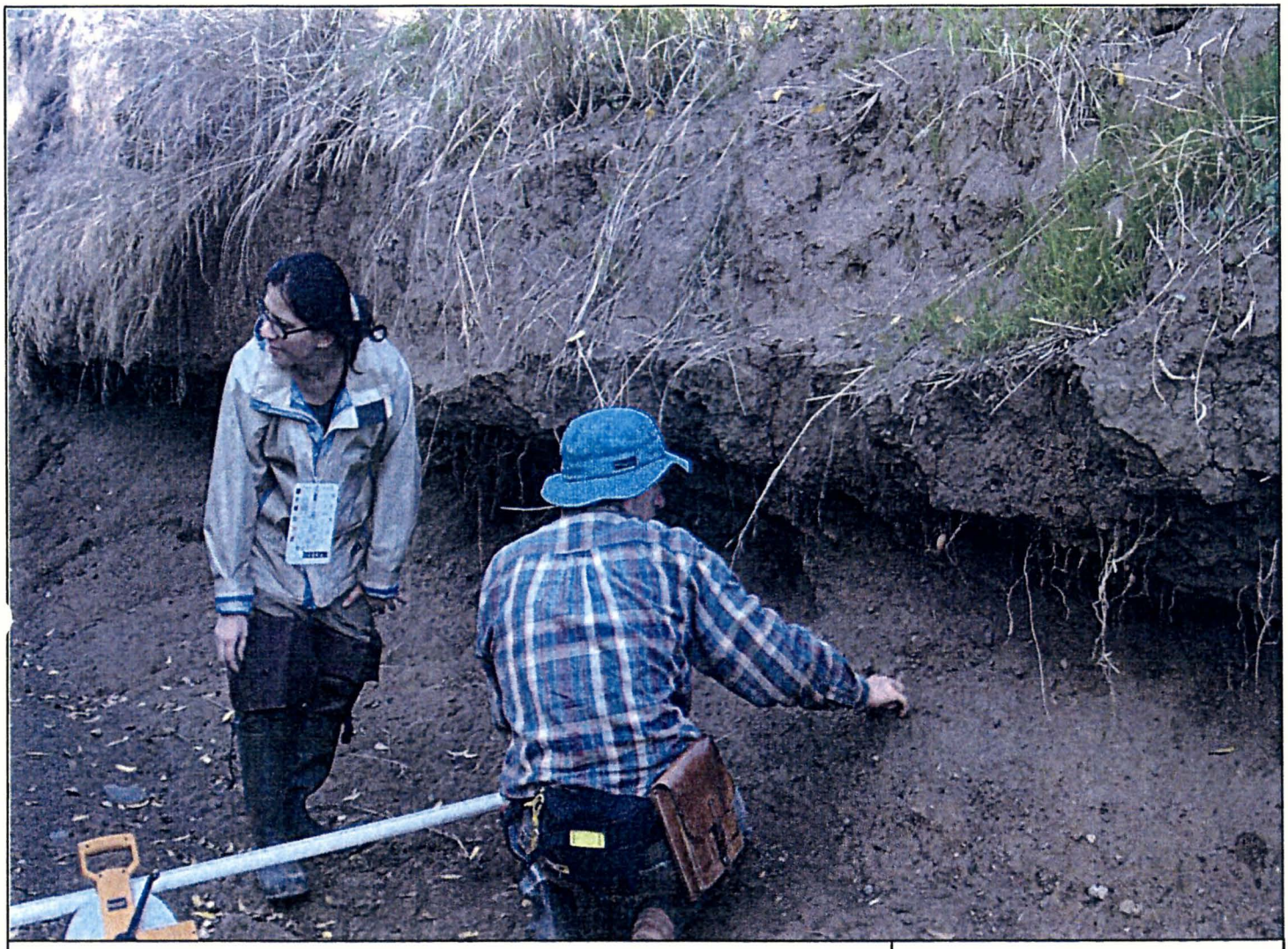
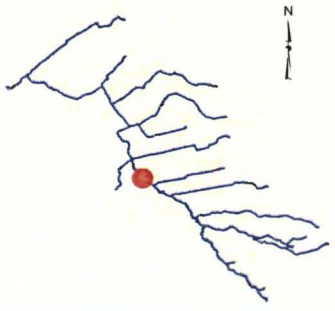


Figure 3-5
Photo of the bank material in Segment TC1, near cross-section TC1-3. Note that the lower unit in the bank erodes more easily than the upper layer.



Cross Section Location

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Figure 3-6
The upper portion of segment TC2 shows less incision than below Aborn Road. The banks are stabilized by tree roots and blackberries, though some bank erosion is still occurring (Photo taken at cross-section TC2-9).

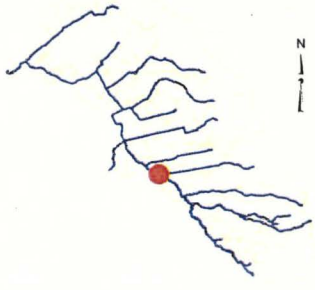




Figure 3-7
Segment TC3 is similar to Segment TC2, but with slightly more incision.



Cross Section Location



Figure 3-8
Photo at cross-section TC5-5. Note the dense vegetation, relatively straight channel, and exposed root mat near water level.



Cross Section Location



Figure 3-9
Photo showing contact between cohesive clayey silt (bottom of bank) and non-cohesive clayey sand and gravel (upper portion of bank). Note that the upper portion of the bank is partially slumped.



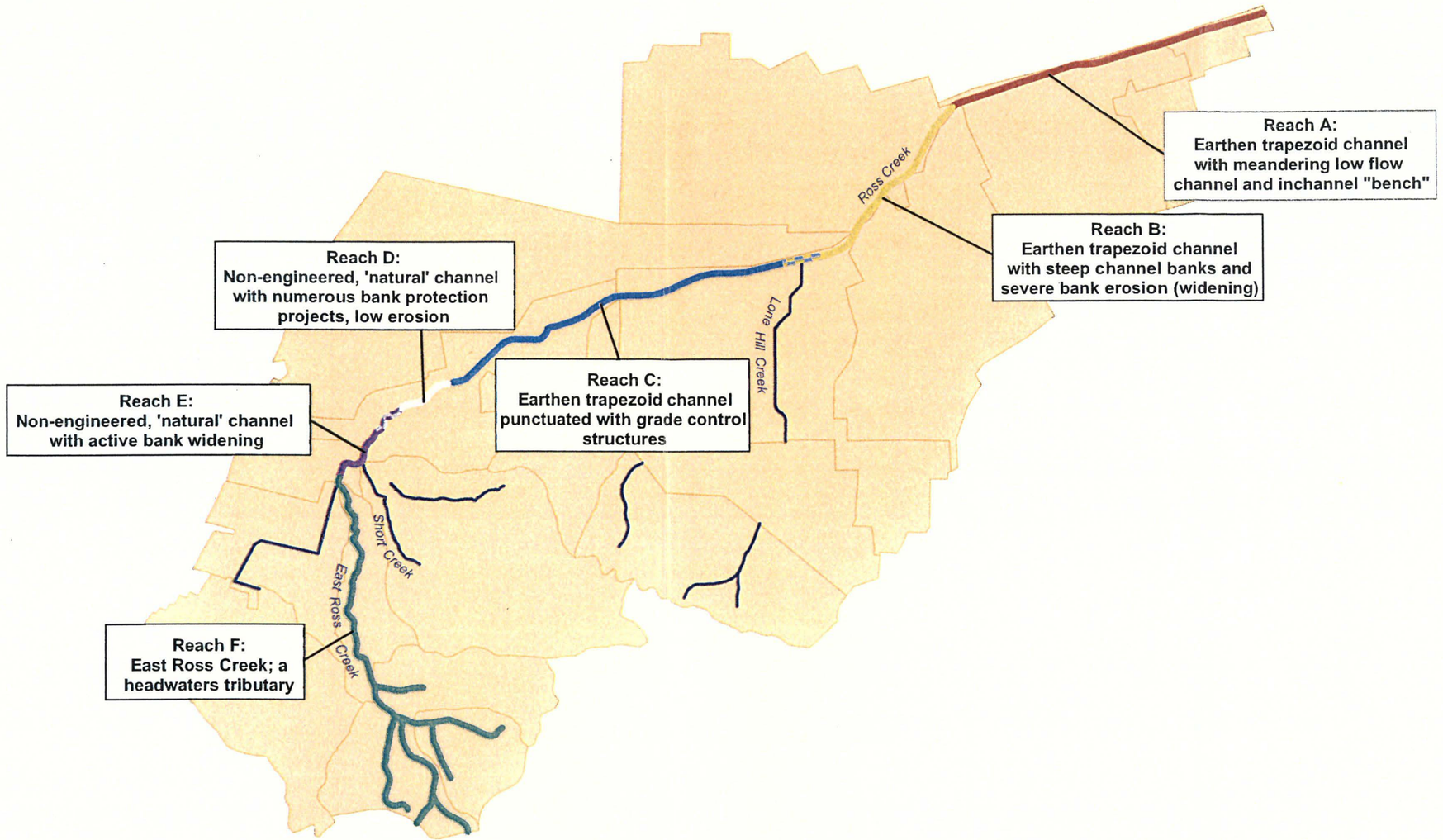
Cross Section Location



Figure 3-10
Photo of deeply incised portion of Yerba Buena Creek at cross-section YB1-6. The creek has incised well below the majority of the roots in the bank.



Cross Section Location



Legend

- Ross Creek Subwatershed
- Major Creek

Geomorphic Reaches

- A
- B
- C
- D
- E
- F
- Transitional Segment

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Figure 3-11
Ross Creek Subwatershed
Geomorphic Reaches: Locations and Descriptions

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Legend

- Cross Section Location
- Problem site
- Outfall
- Creek
- Main
- Lateral
- Catchment Boundary
- Watershed Boundary

N

0 2,500 5,000 Feet

Figure 3-12
Ross Creek Subwatershed
SCVWD Problem Sites and
Cross Section Locations

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Legend

- Cross Section Location
- San Tomas Outfall
- Creek
- Main
- Lateral
- Subwatershed Boundary

N

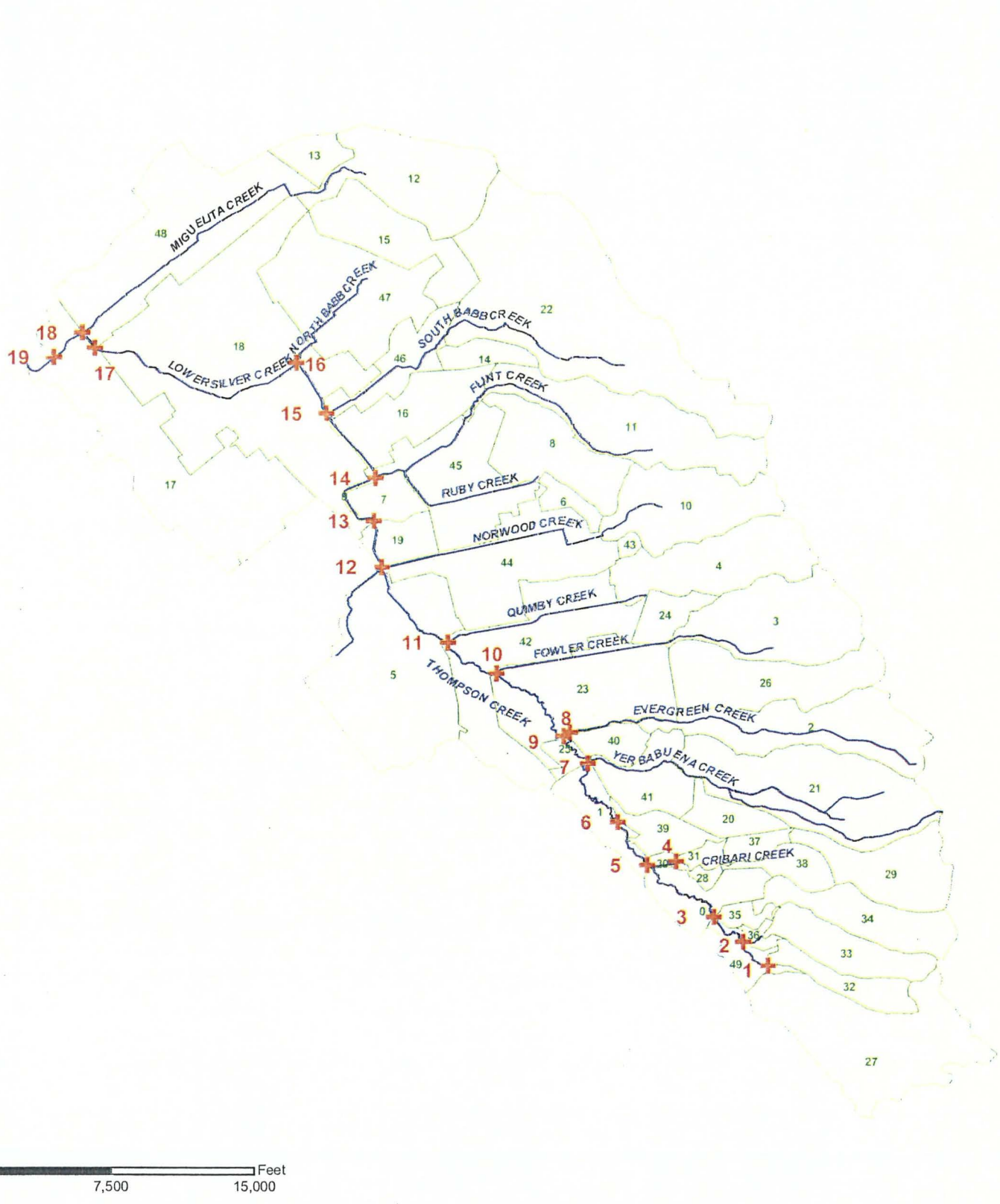
0 Feet 4,000

Figure 3-13
San Tomas Creek Subwatershed
SCVWD Problem Sites and
Cross Section Locations

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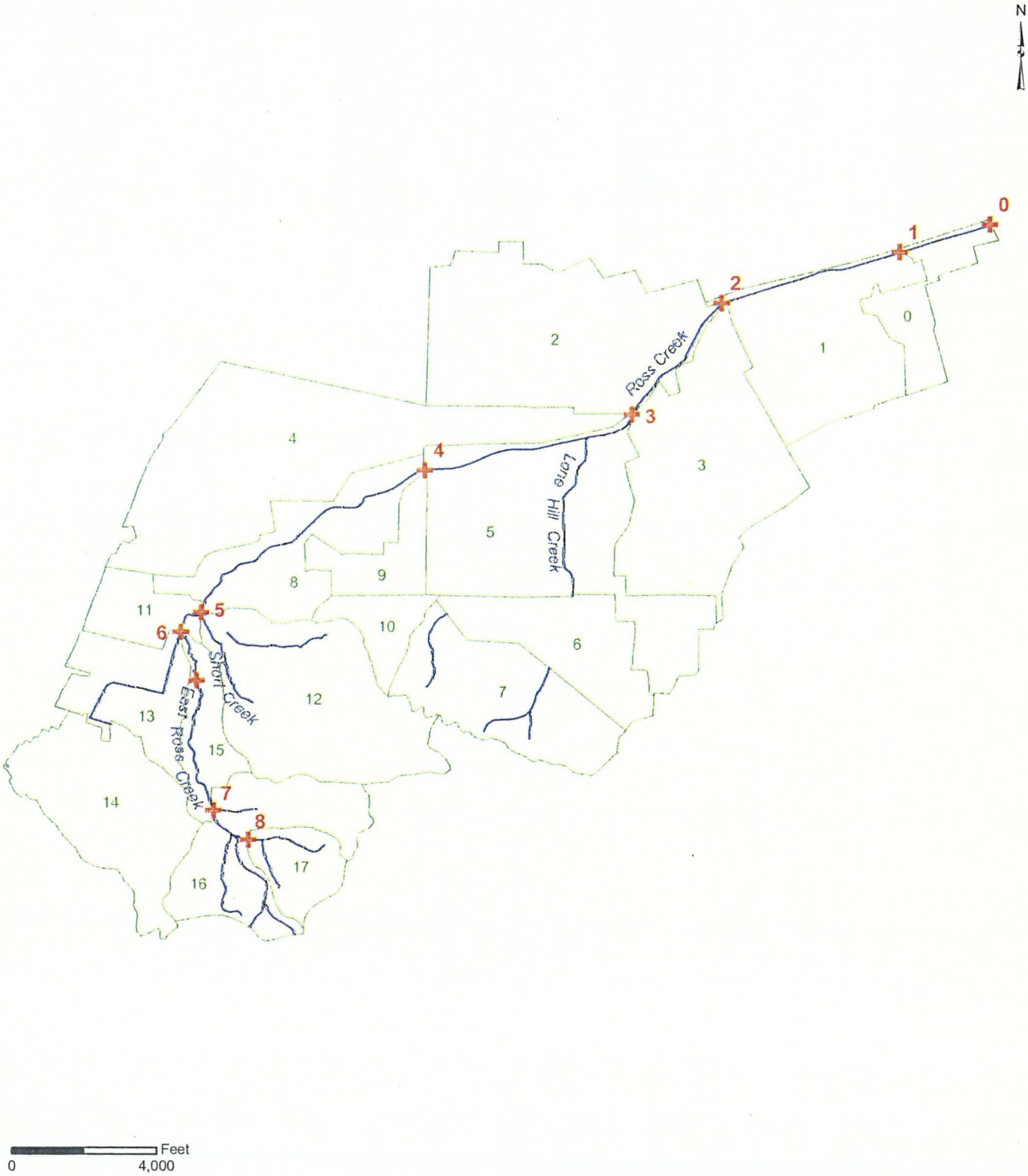
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Legend

- Junction Locations and Names
- Drainage Areas
- Main Roads

Figure 4-1
Drainage Area Delineations for the
Lower Silver - Thompson Creek Subwatershed
(Note: Creeks have been shown above ground but actually run underground in some of the urban areas)

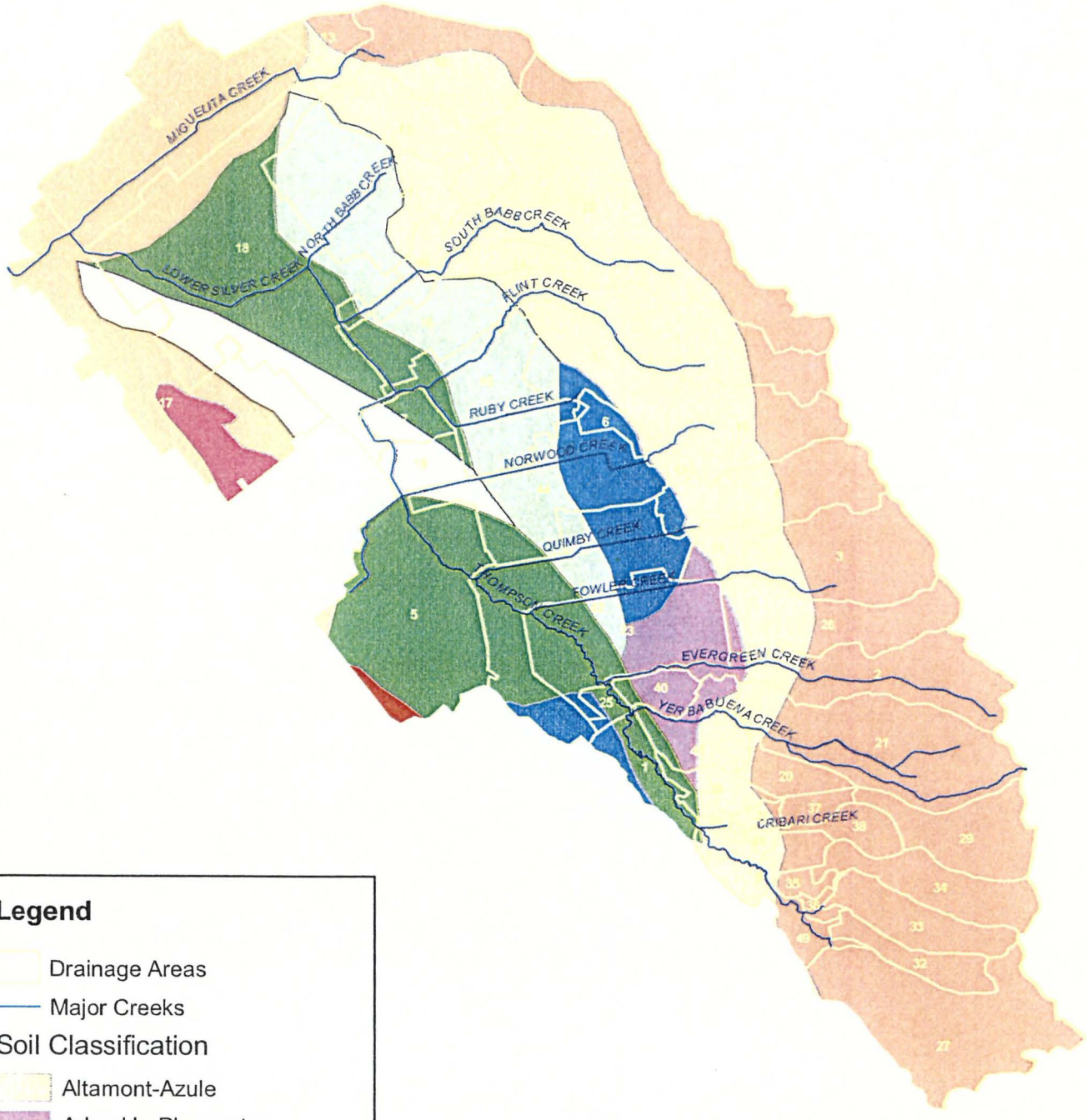


Legend

- Junction Location
- Drainage Area
- Major Creek

Figure 4-2
Drainage Area Delineations for the Ross Creek Subwatershed

(Note: Creeks have been shown above ground but actually run underground in some of the urban areas)



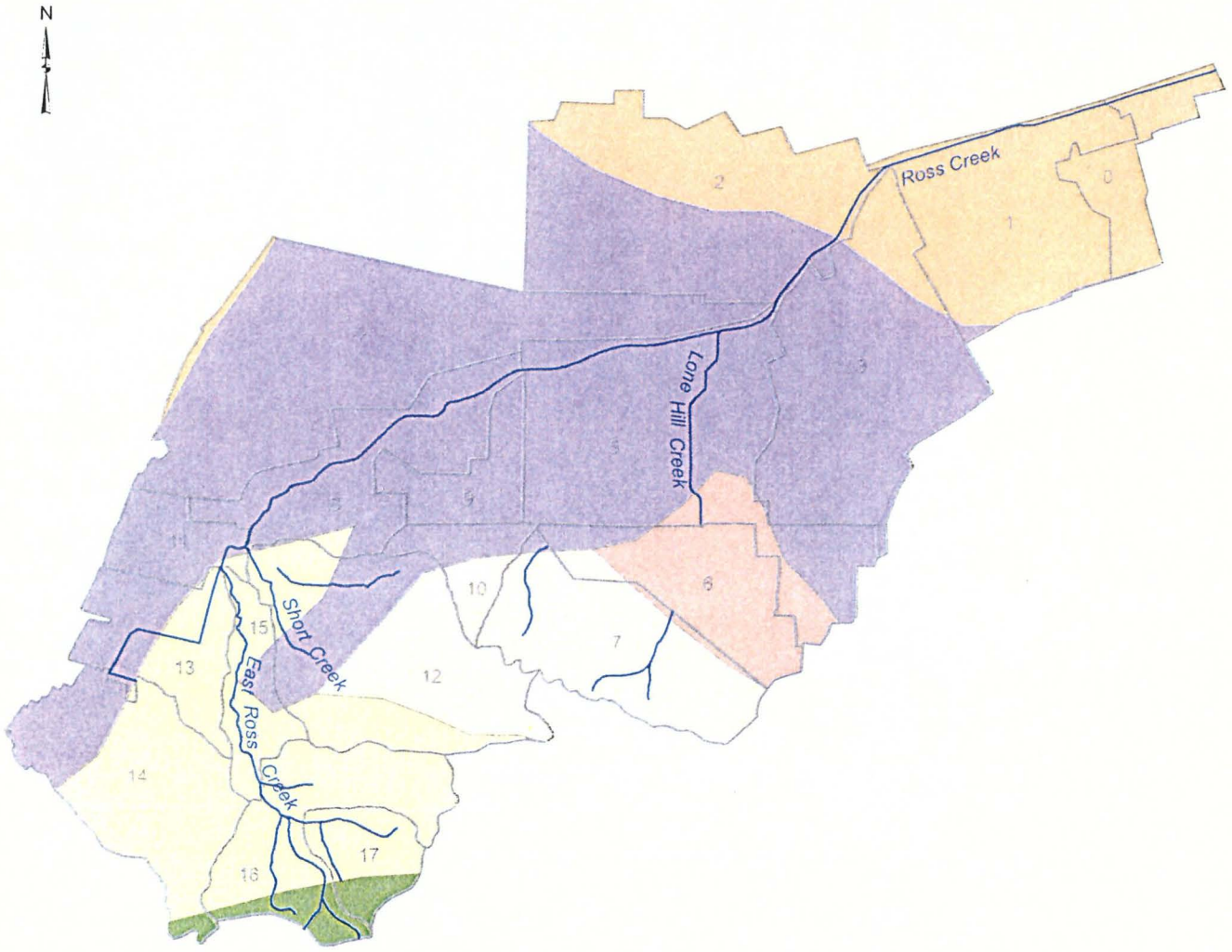
Legend

- Drainage Areas
- Major Creeks

Soil Classification

- Altamont-Azule
- Arbuckle-Pleasanton
- Clear Lake-Campbell
- Cropley-Rincon
- Hillgate-Positas
- Los Gatos-Gaviota-Vallecitos
- Orestimba-Willows
- Sunnyvale-Castro-Clear Lake
- Yolo
- Zamora-Pleasanton

Figure 4-3
Soil Classifications for the
Lower Silver - Thompson Creek Subwatershed



Legend

- Drainage Area
- Major Creek

Soil Classification

- Altamont-Azule
- Arbuckle-Pleasanton
- Clear Lake-Campbell
- Los Gatos-Gaviota-Vallecitos
- Maymen-Los Gatos-Gaviota
- Yolo

0 2,000 4,000 Feet

Figure 4-4
Soil Classifications for the Ross Creek Subwatershed

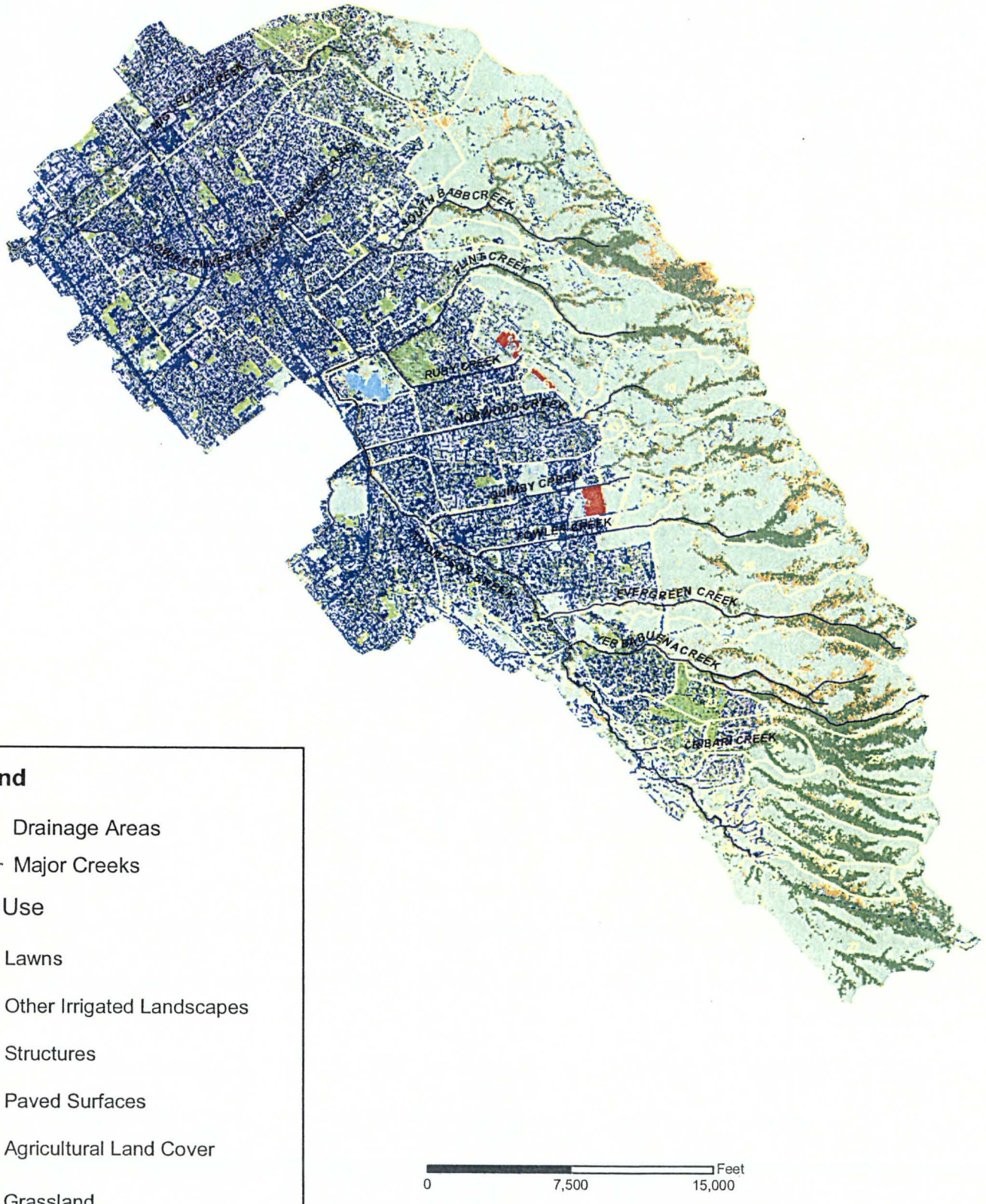


Figure 4-5
Land Use for the
Lower Silver - Thompson Creek Subwatershed

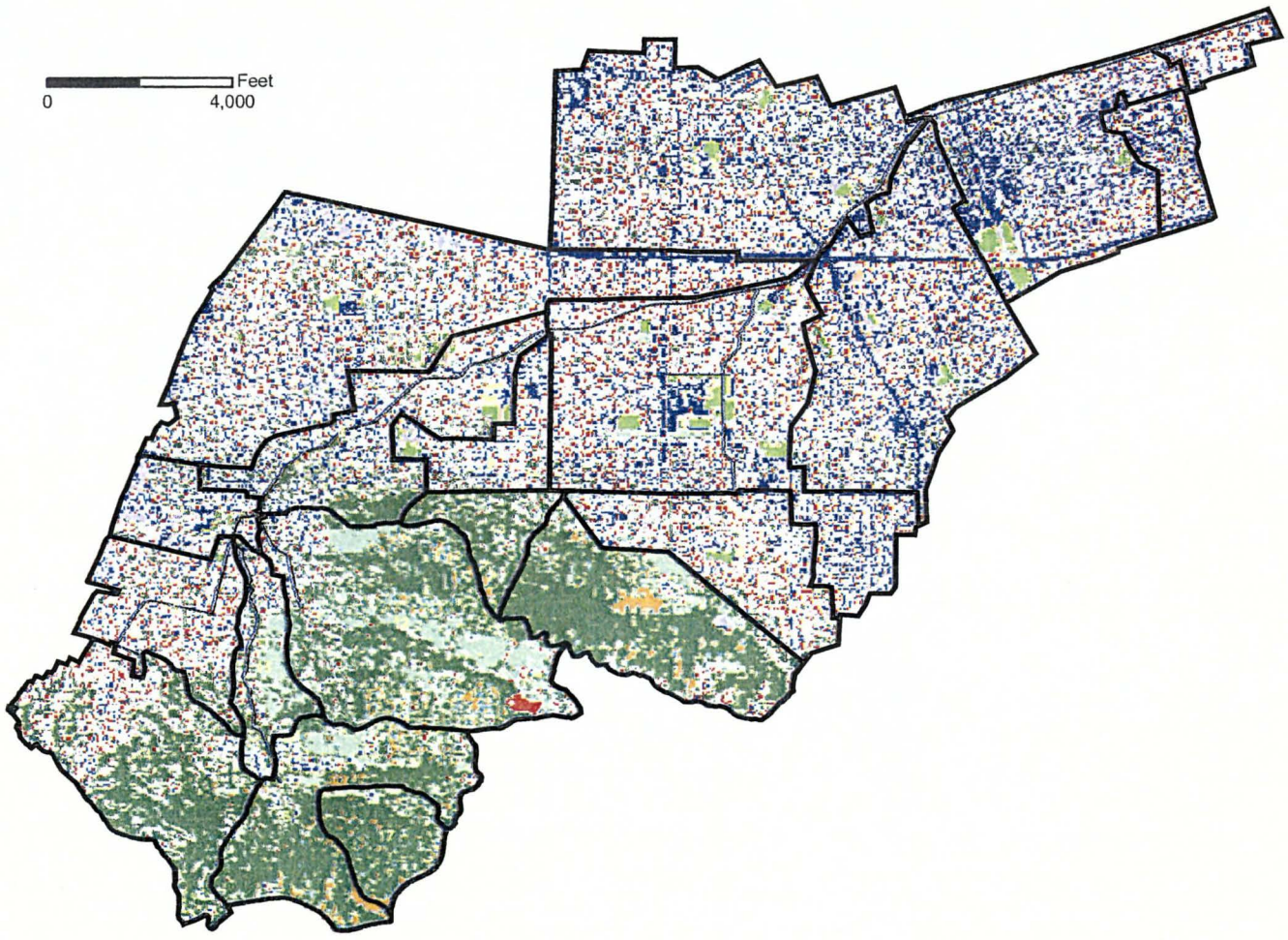


Figure 4-6
Land Use for the Ross Creek Subwatershed

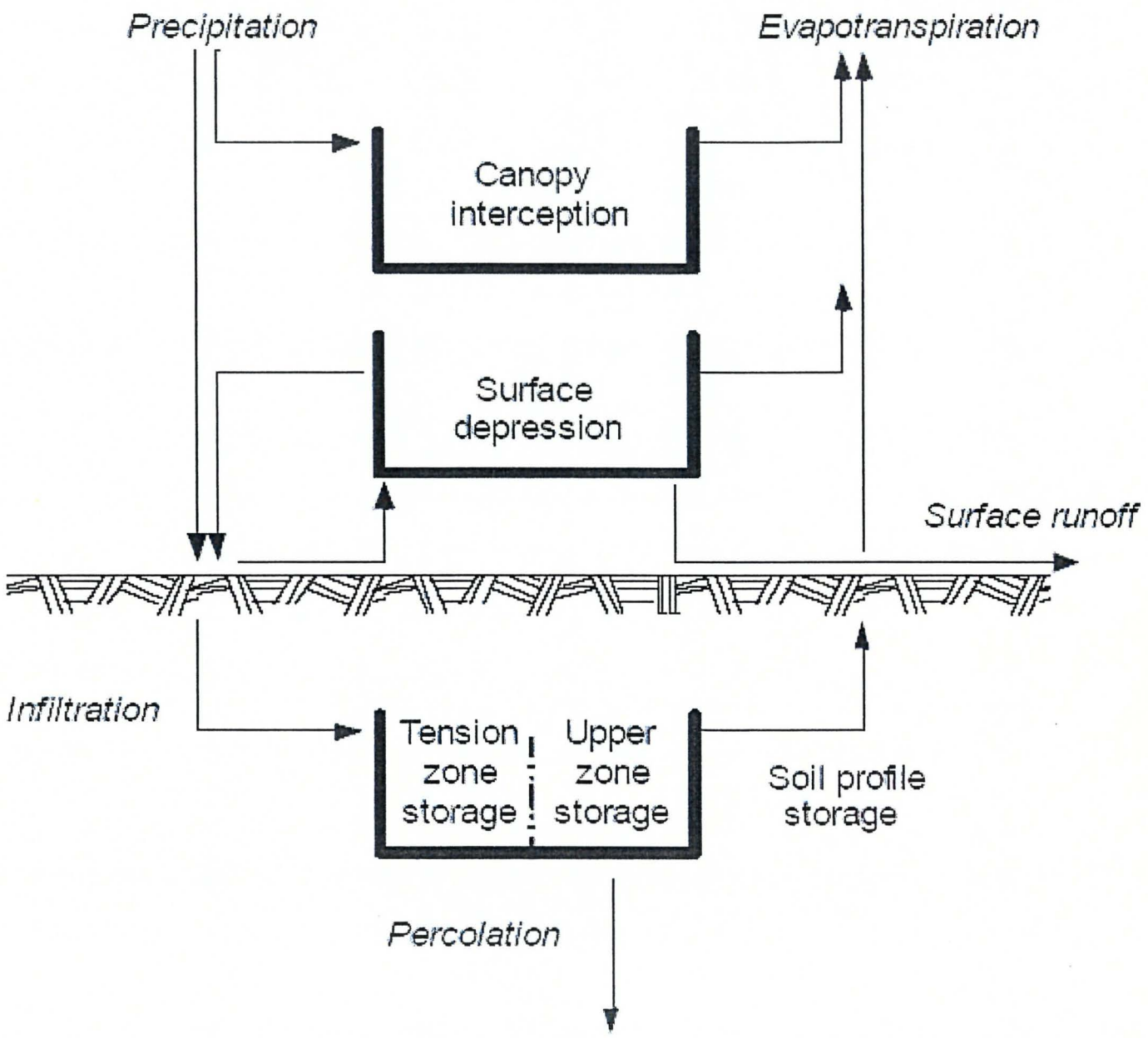
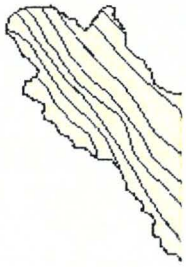
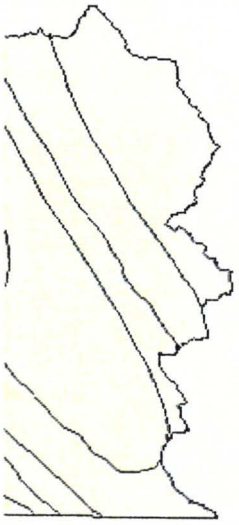


Figure 4-7
Schematic of HEC-HMS Soil Moisture Accounting Model





0 30,000 60,000 Feet

Legend

- Rain Gage
- Ross Creek Subwatershed
- Thompson-Lower Silver Creeks Subwatershe
- Surrounding Santa Clara County
- Baylands
- Isopleth

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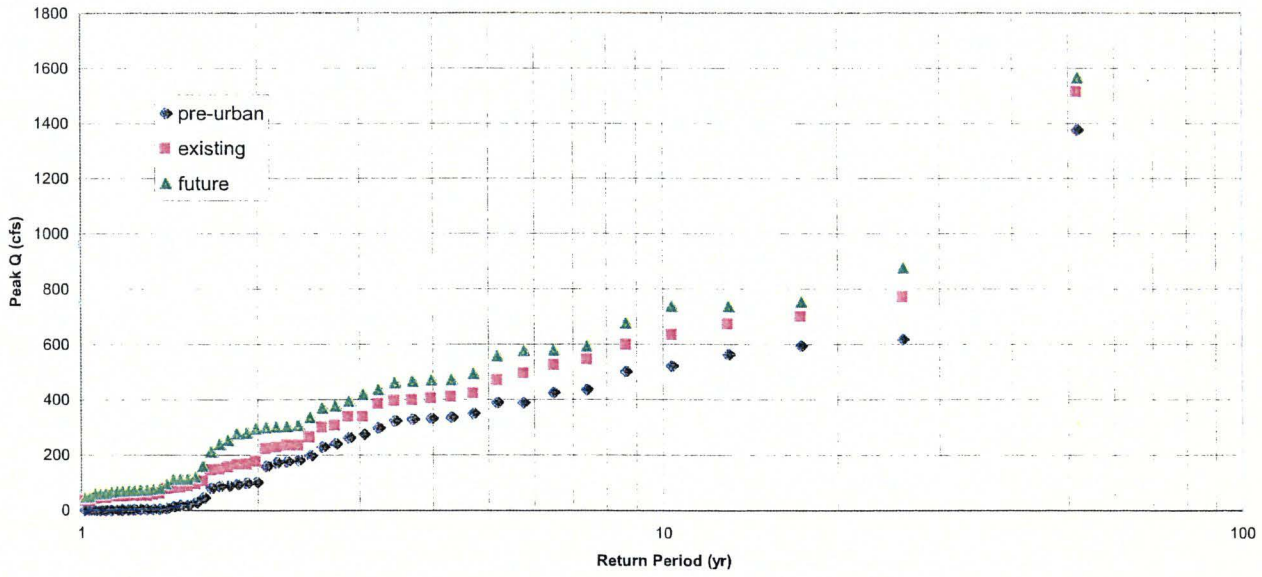
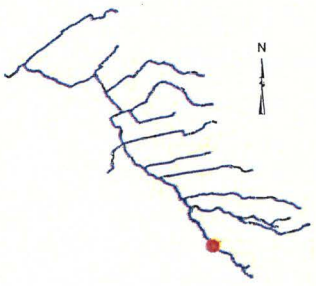


Figure 4-9
Flood Frequency for Junction J-5
Using Continuous Model Results



Junction Location

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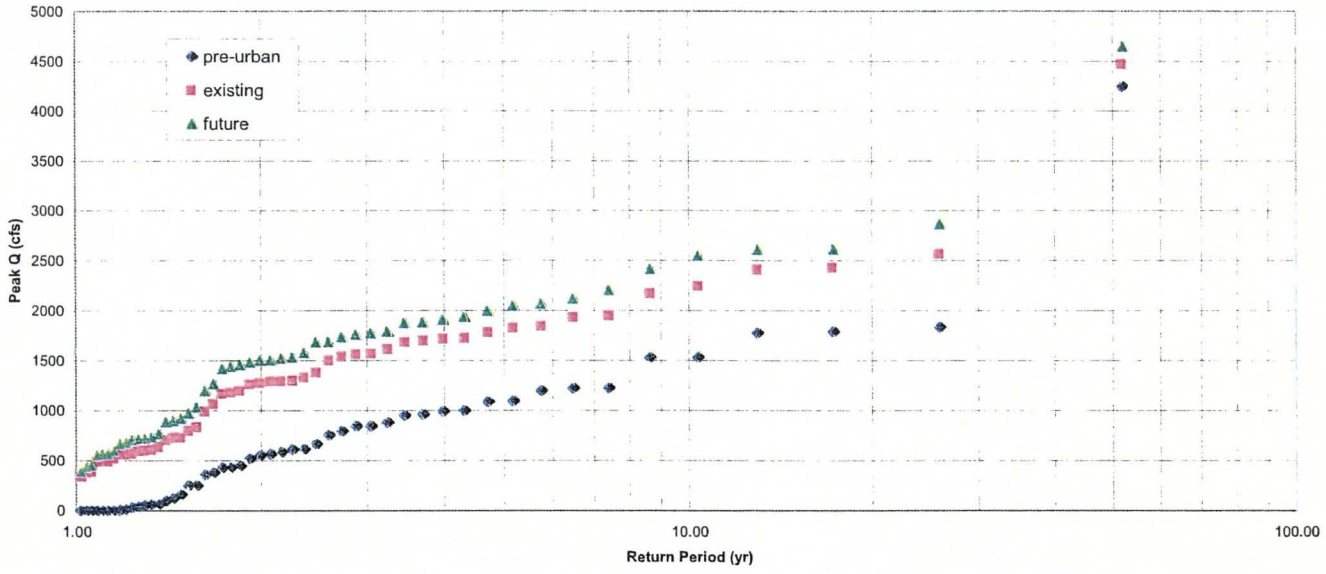
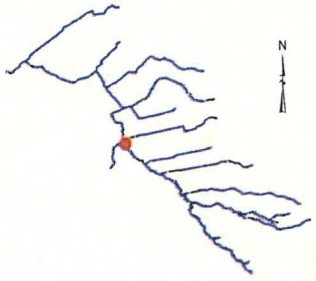


Figure 4-10
Flood Frequency for Junction J-12
Using Continuous Model Results



Junction Location

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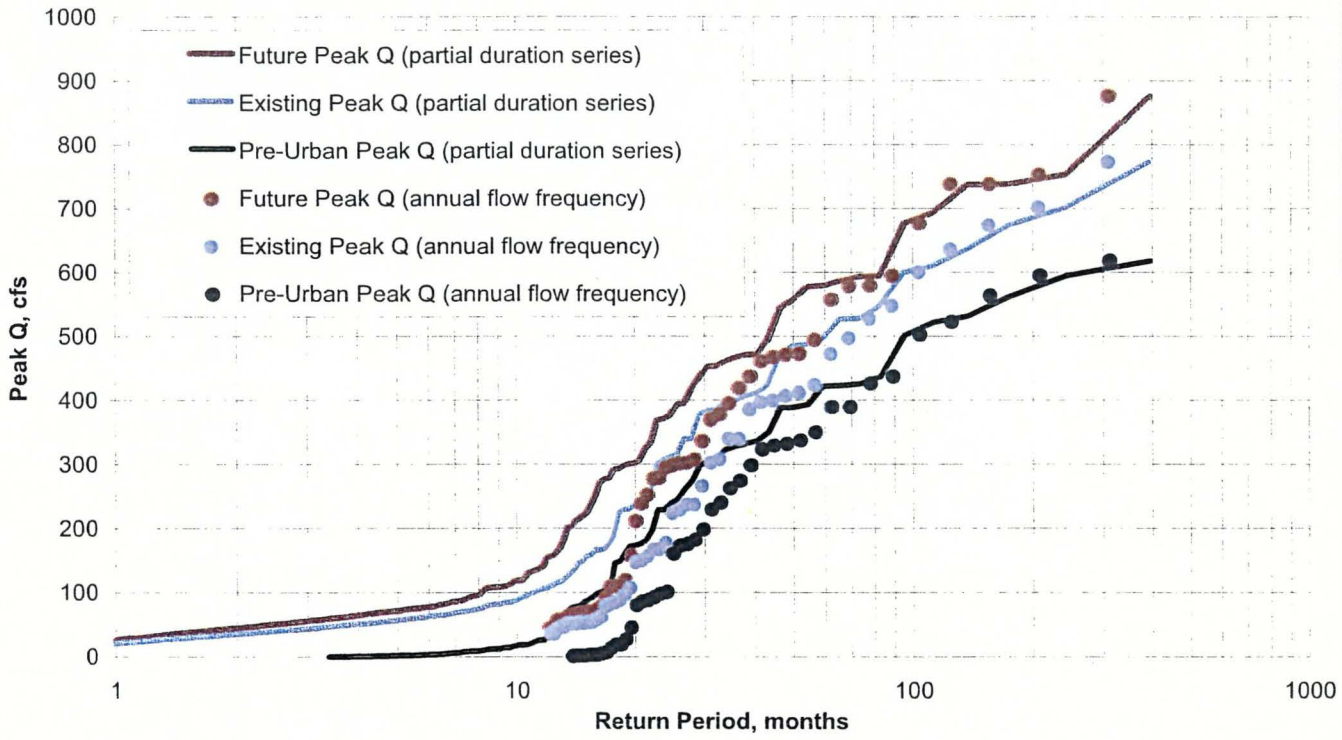
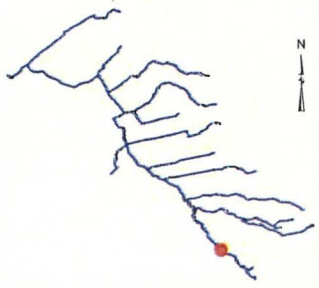


Figure 4-11
Peak Flow Frequency Analysis: Monthly and Annual Recurrence Intervals at Location J-5



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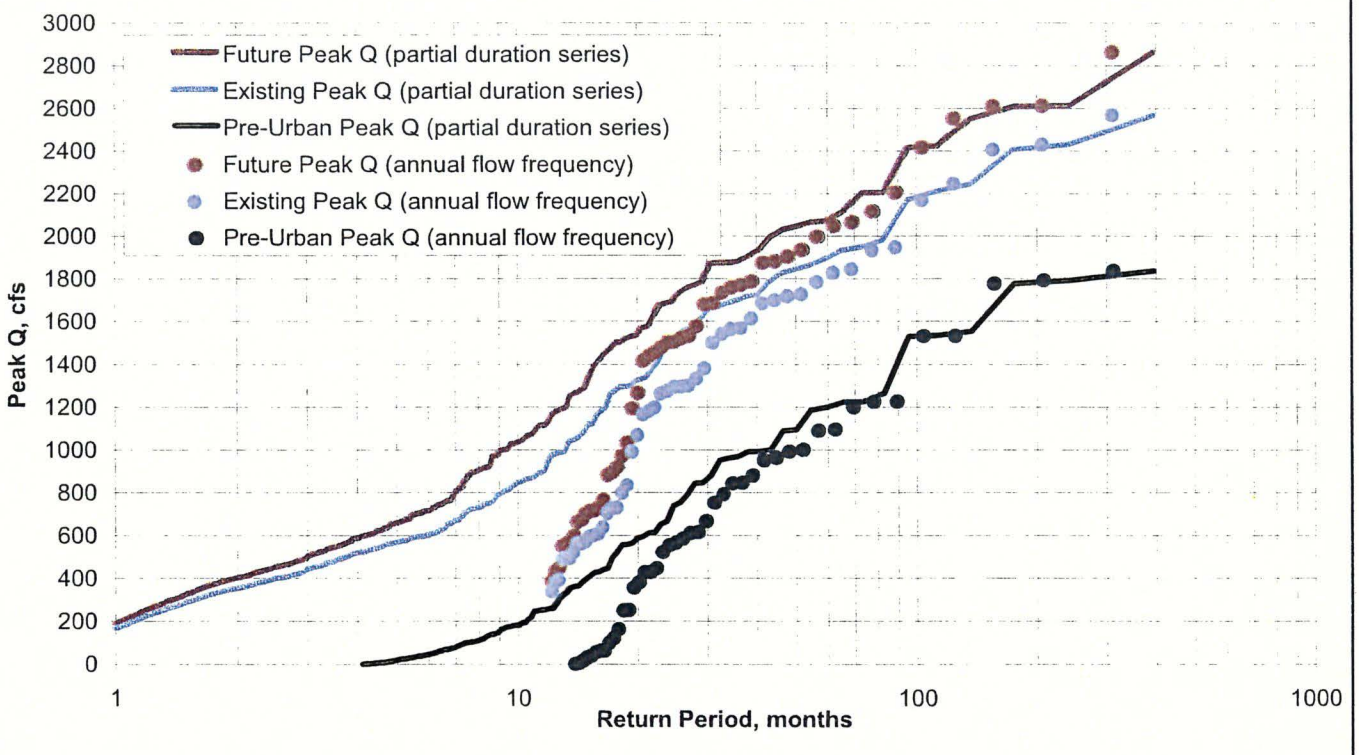
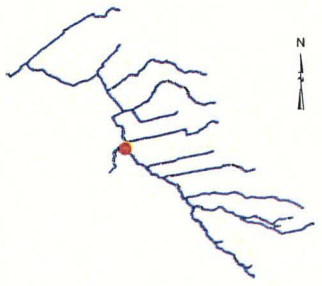


Figure 4-12
Peak Flow Frequency Analysis: Monthly and Annual Recurrence Intervals at Location J-12



Junction Location

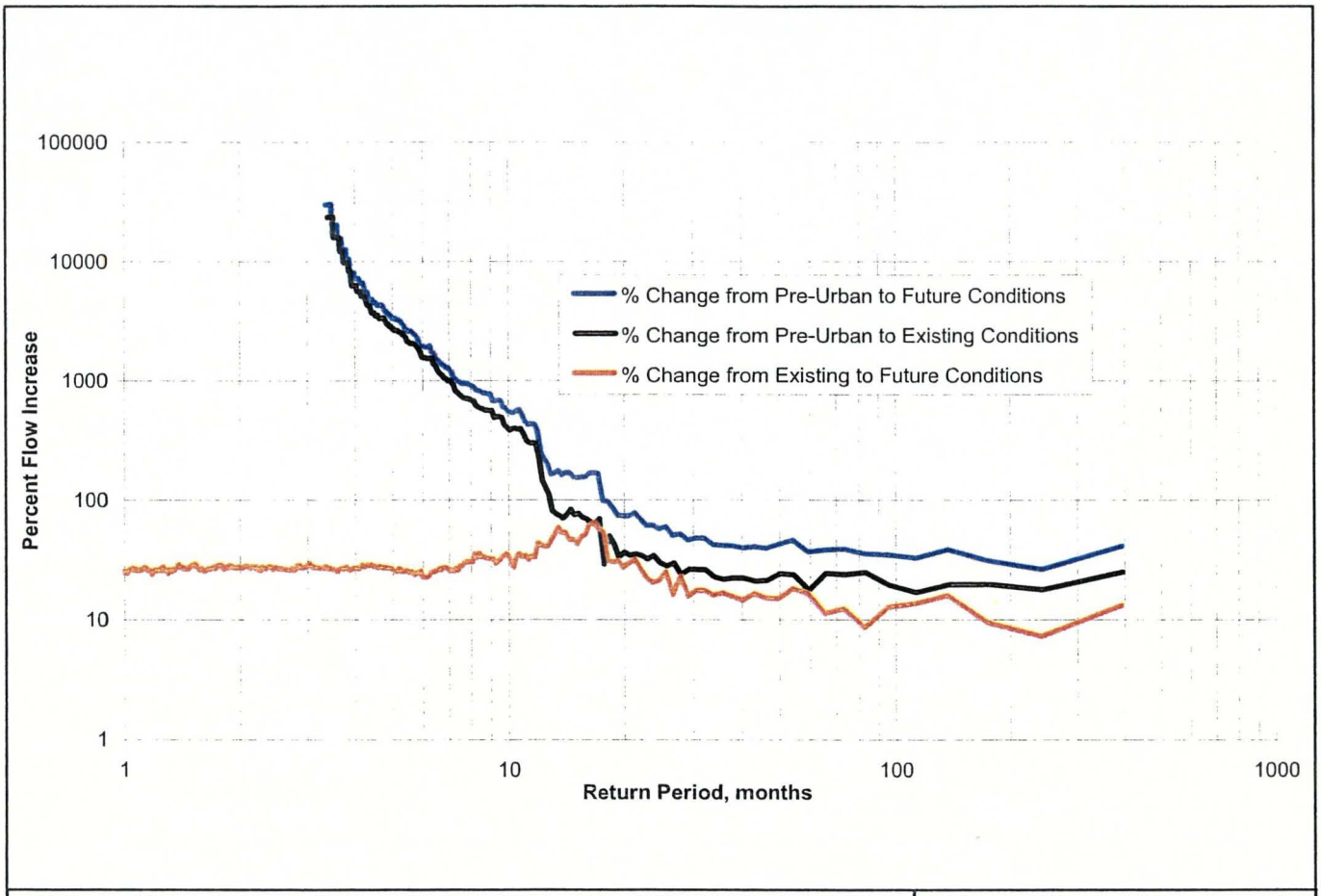
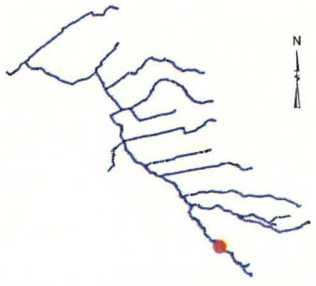


Figure 4-13
Percent Flow Increase at Location J-5



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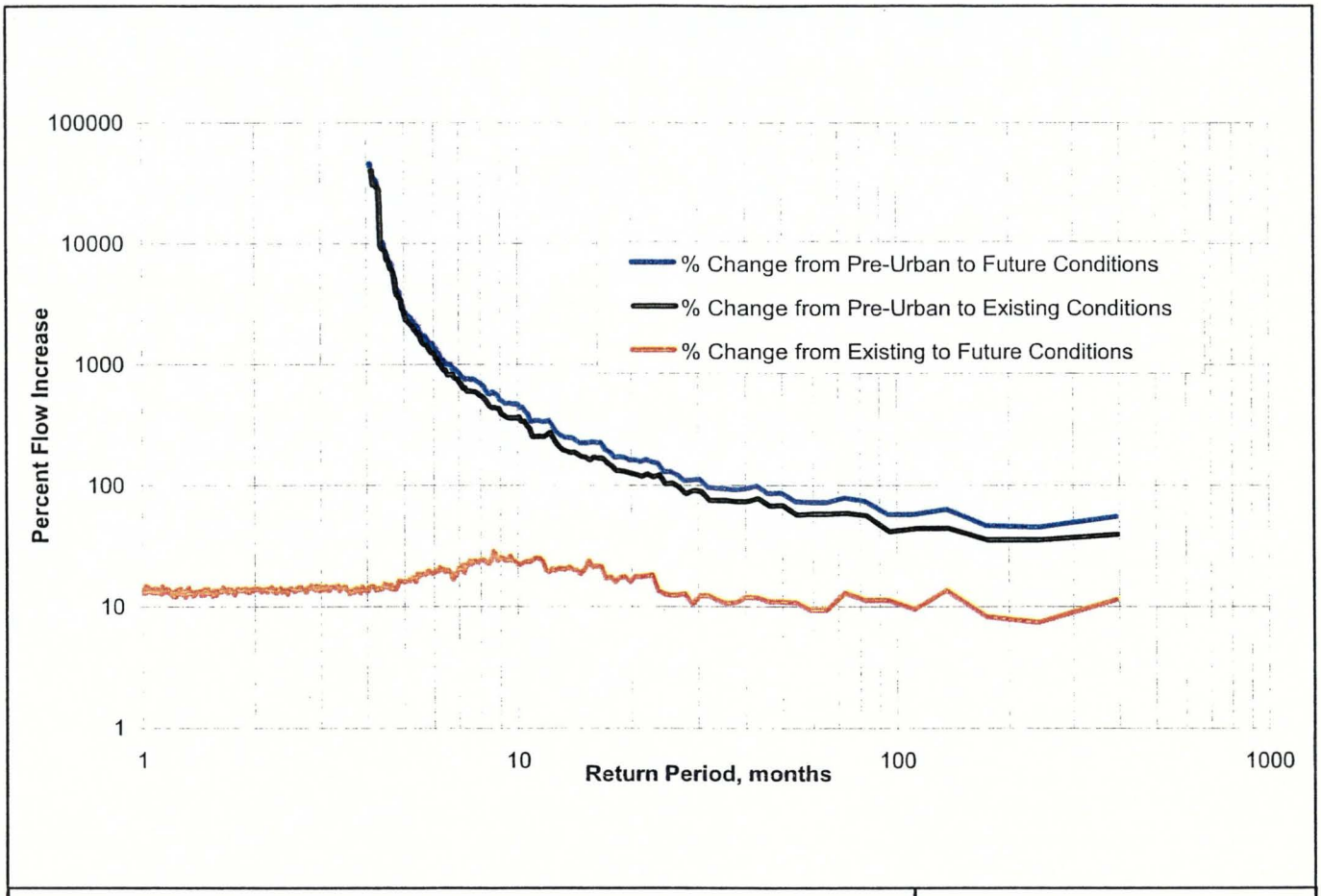
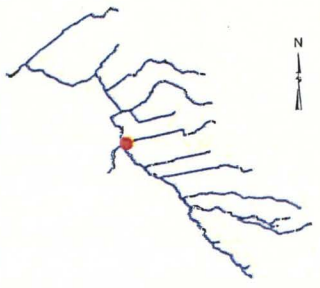


Figure 4-14
Percent Flow Increase at Location J-12



Junction Location

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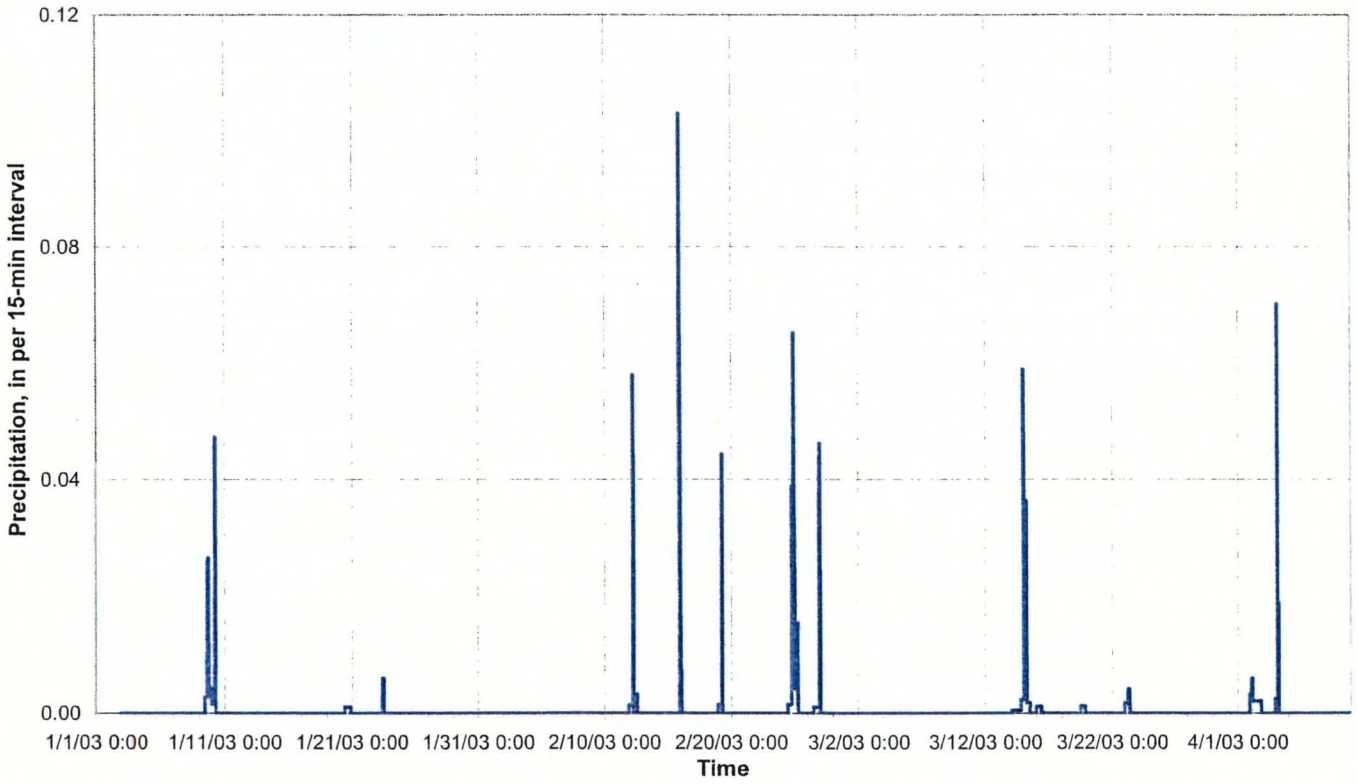


Figure 4-15
Spring 2003 Precipitation

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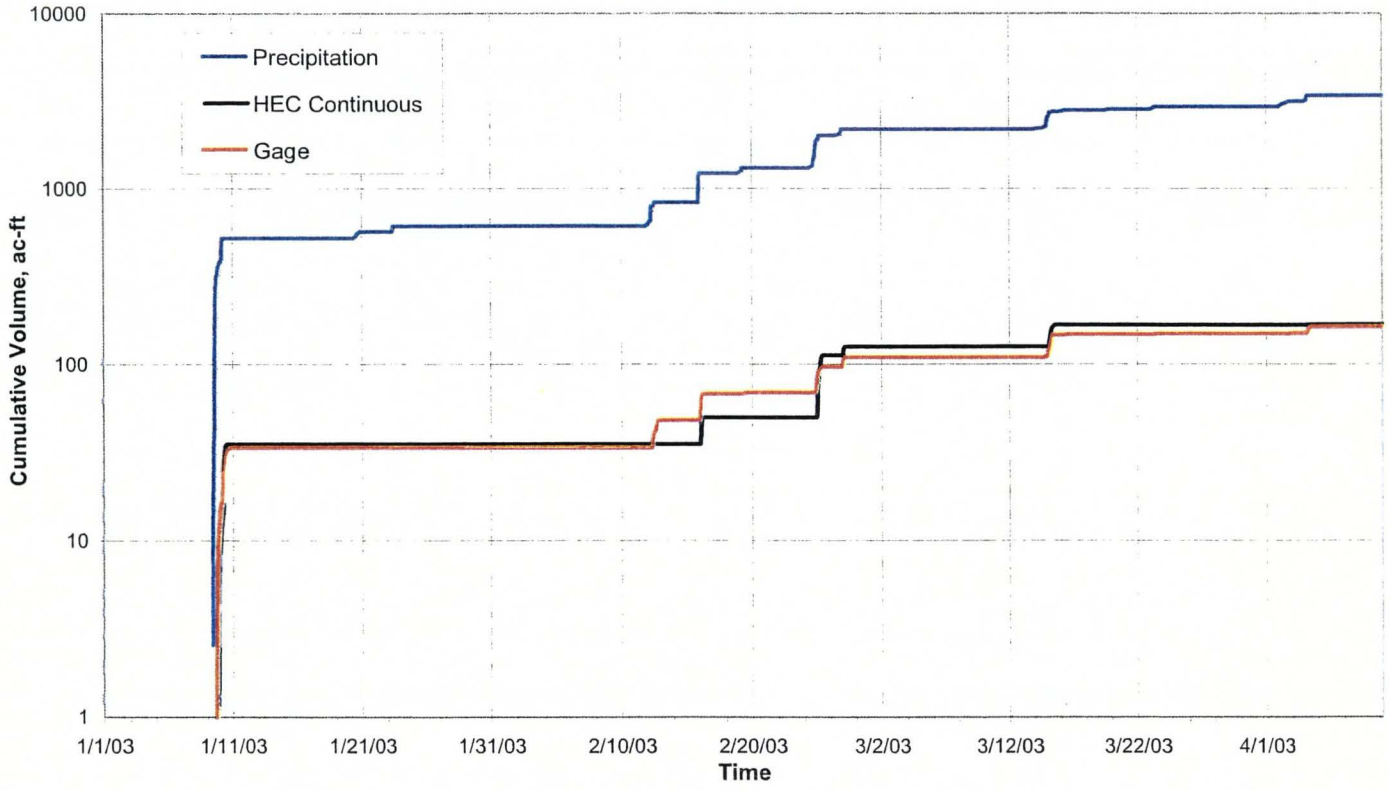


Figure 4-16
Cumulative Volume of Discharge -
Thompson Creek at Quimby Road

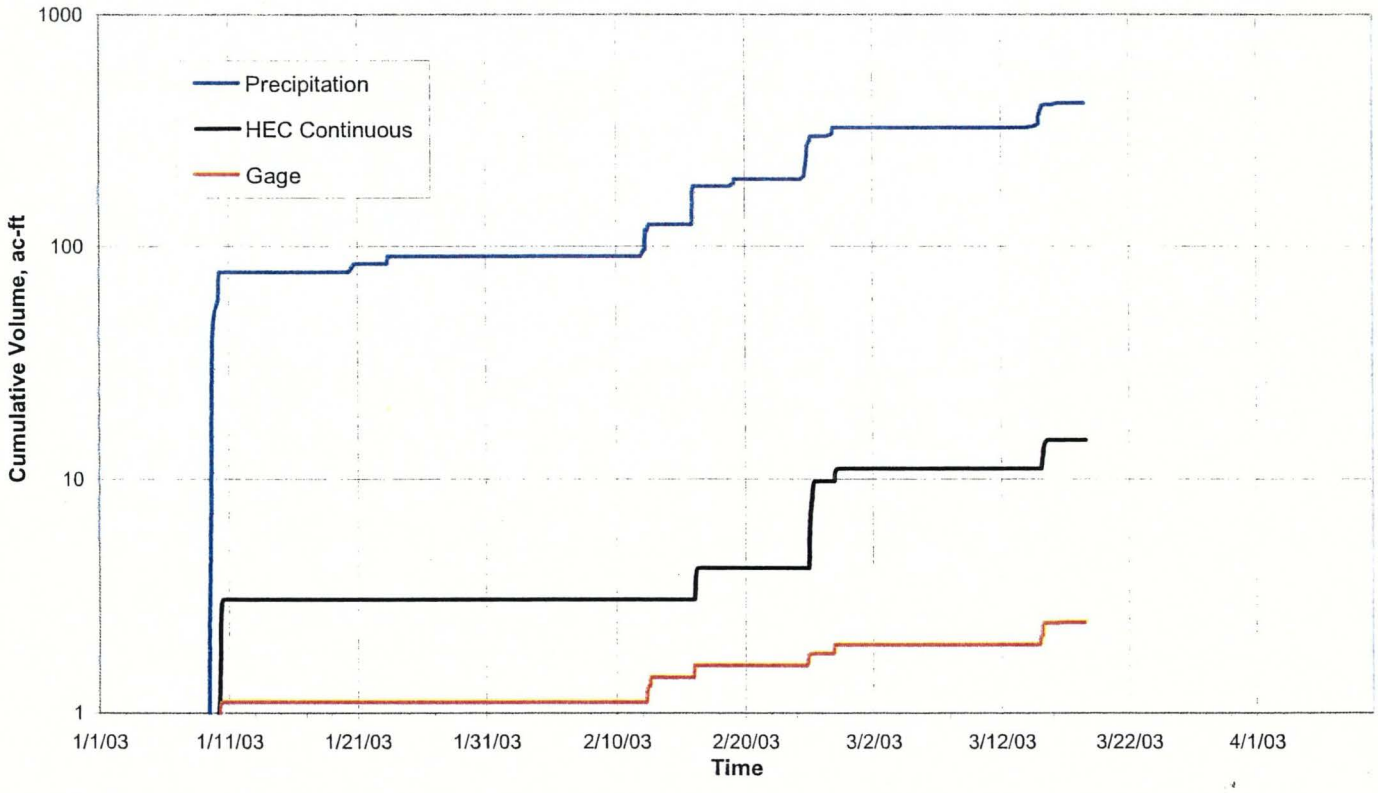


Figure 4-17
Cumulative Volume of Discharge -
Yerba Buena Creek Watershed

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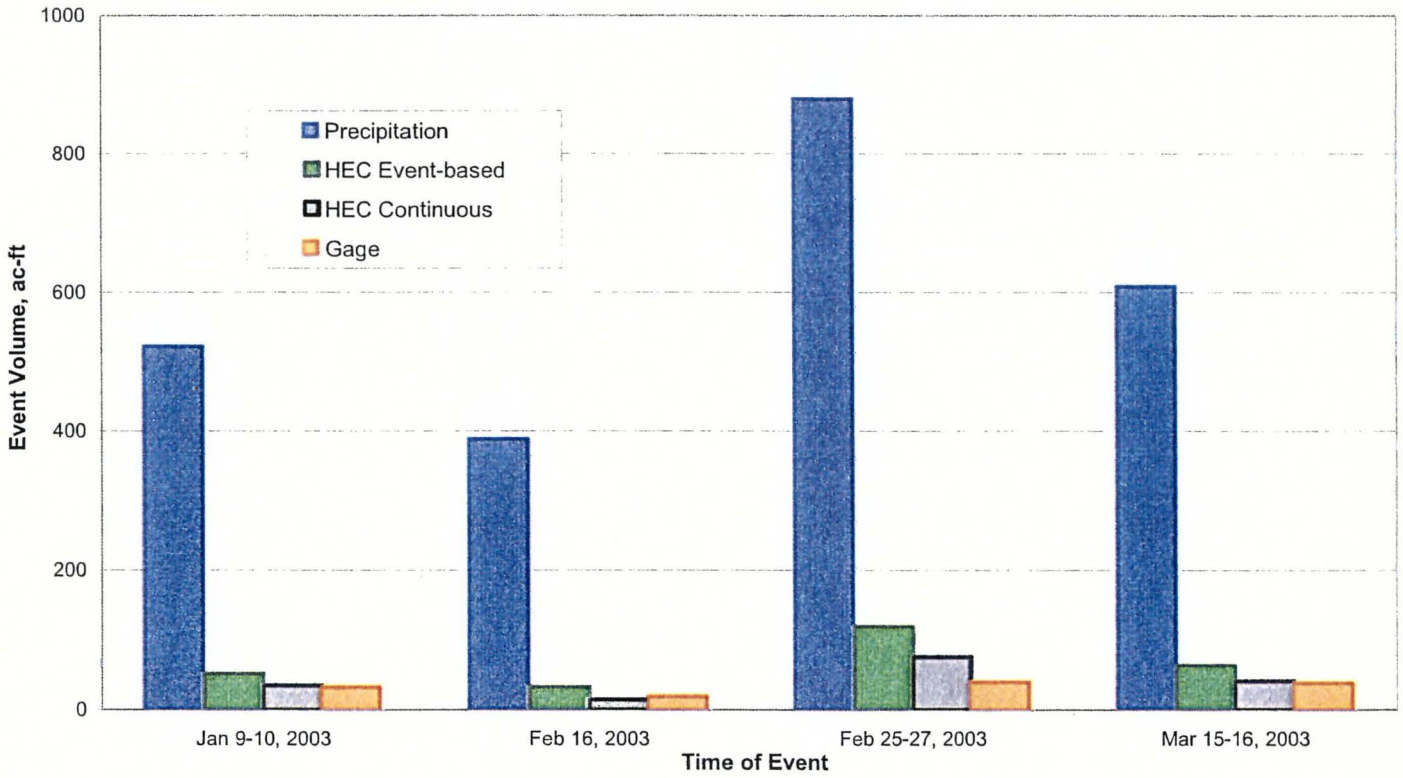


Figure 4-18
Event Volume Comparisons-
Thompson Creek at Quimby Road

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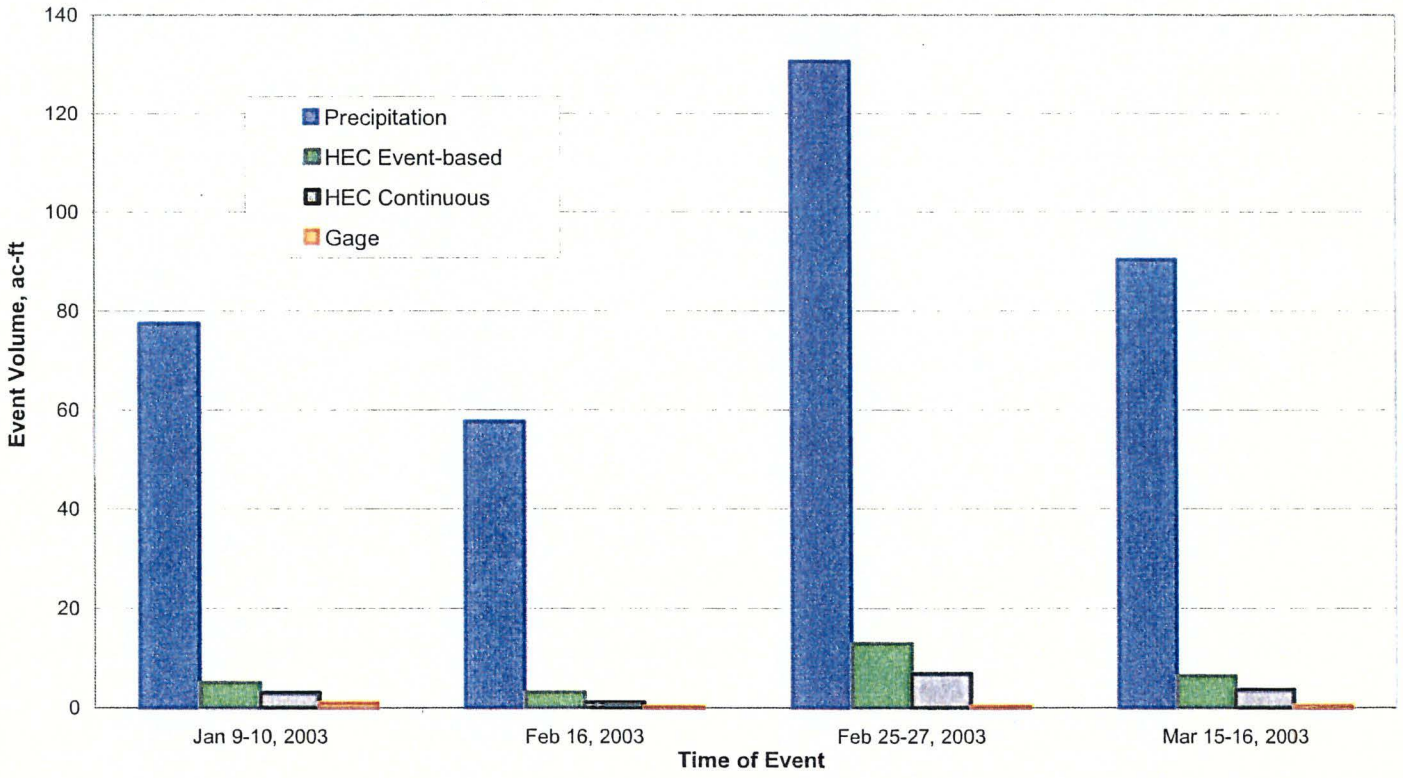


Figure 4-19
Event Volume Comparisons-
Yerba Buena Creek Watershed

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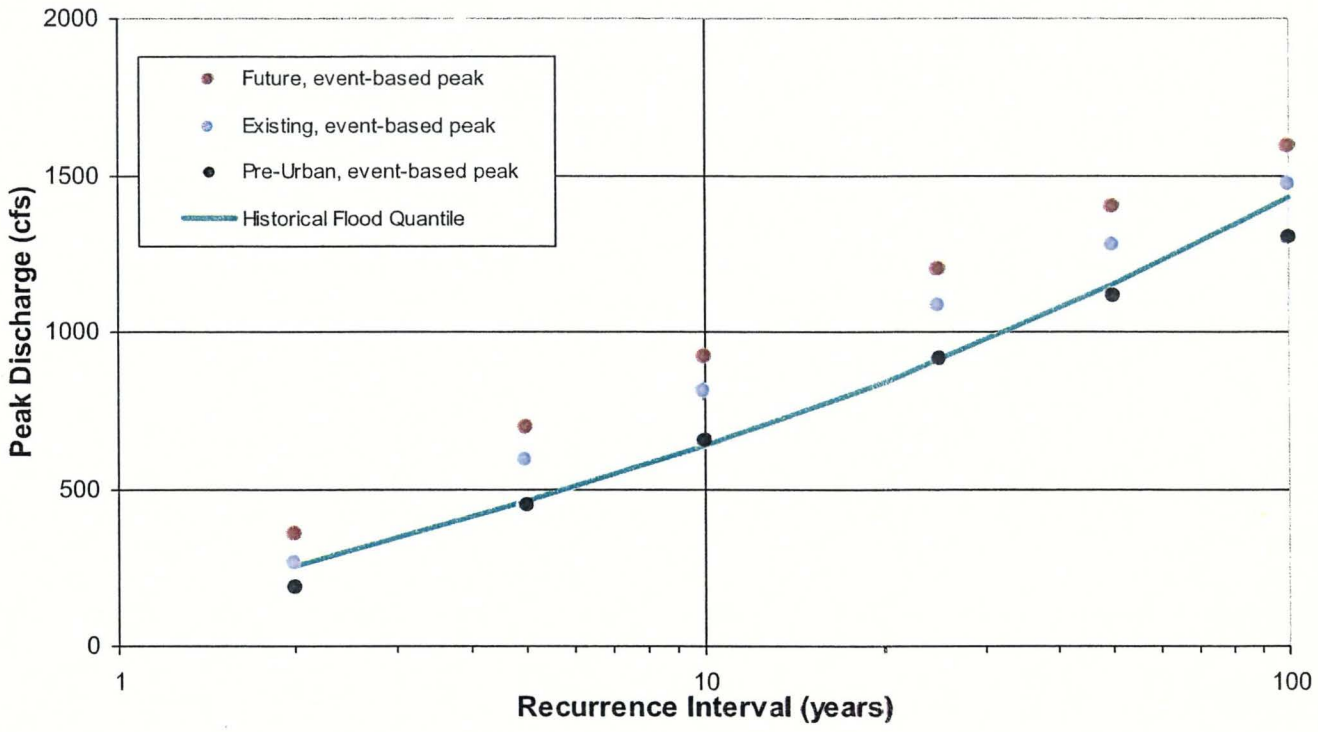
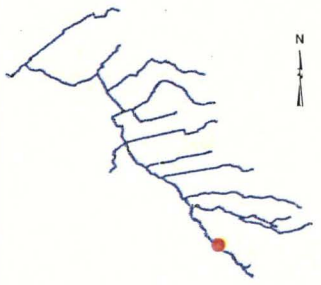


Figure 4-20
Thompson Creek Peak Discharge Recurrence Intervals, Event-Based Model at Junction J-5



Junction Location

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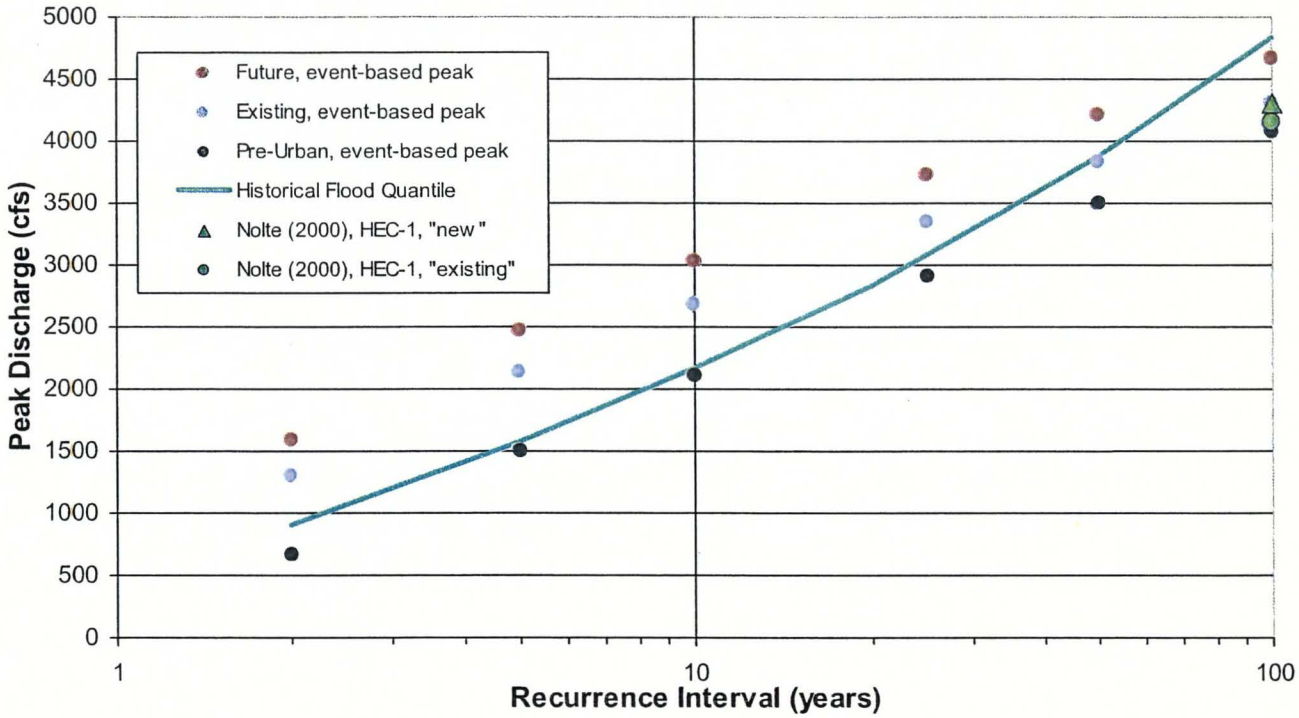
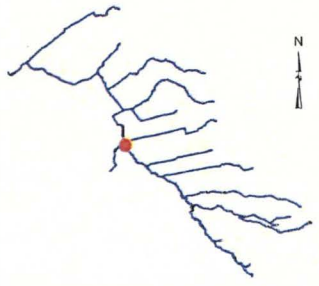


Figure 4-21
Thompson Creek Peak Discharge Recurrence Intervals, Event-Based Model at Junction J-12



Junction Location

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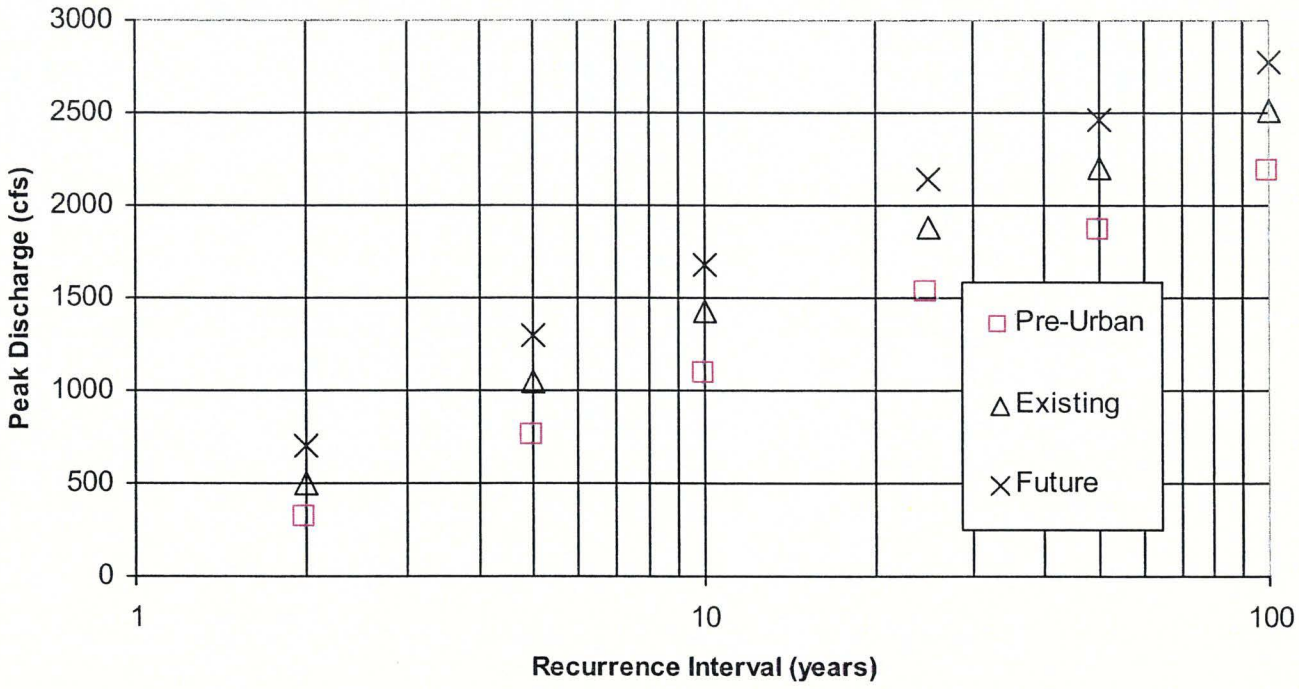
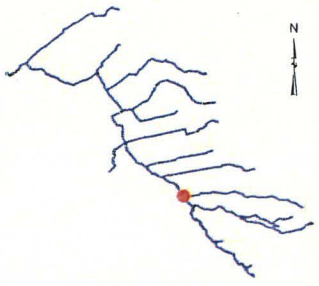


Figure 4-22
Thompson Creek Flood Frequency for Junction J-9 Using Event-Based Model Results



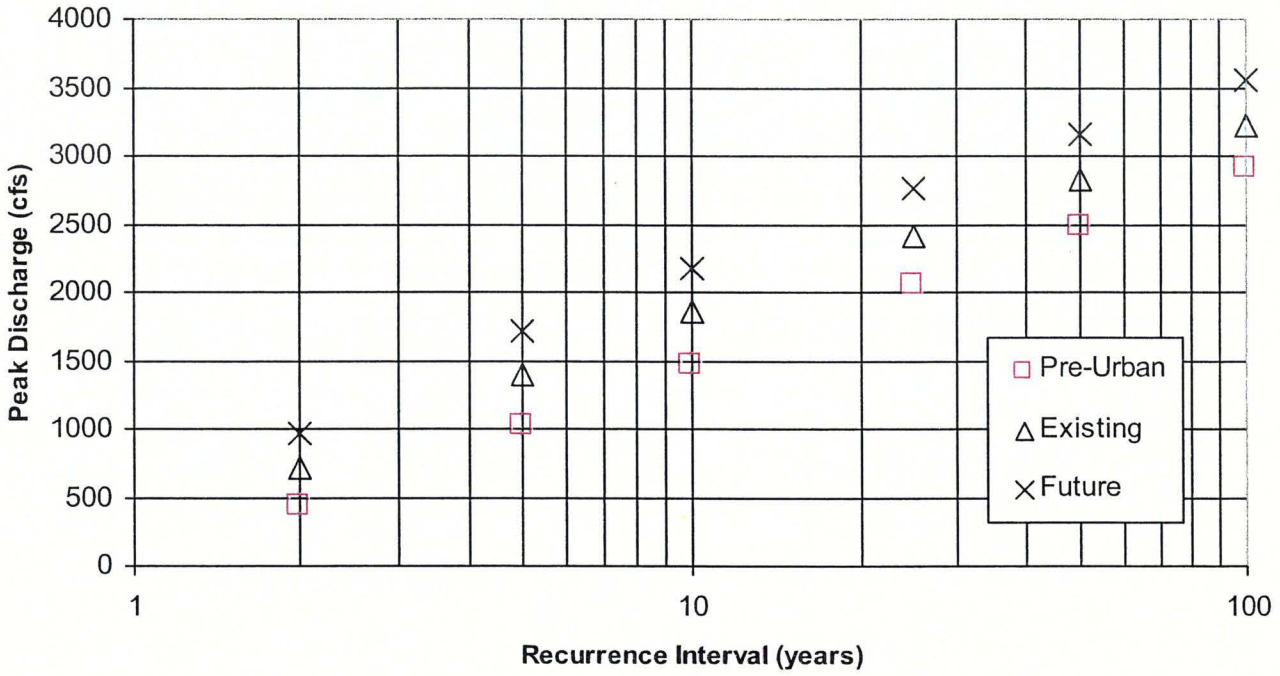
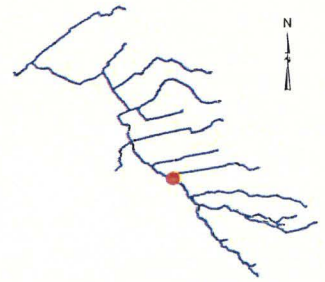


Figure 4-23
Thompson Creek Flood Frequency for Junction J-10 Using Event-Based Model Results



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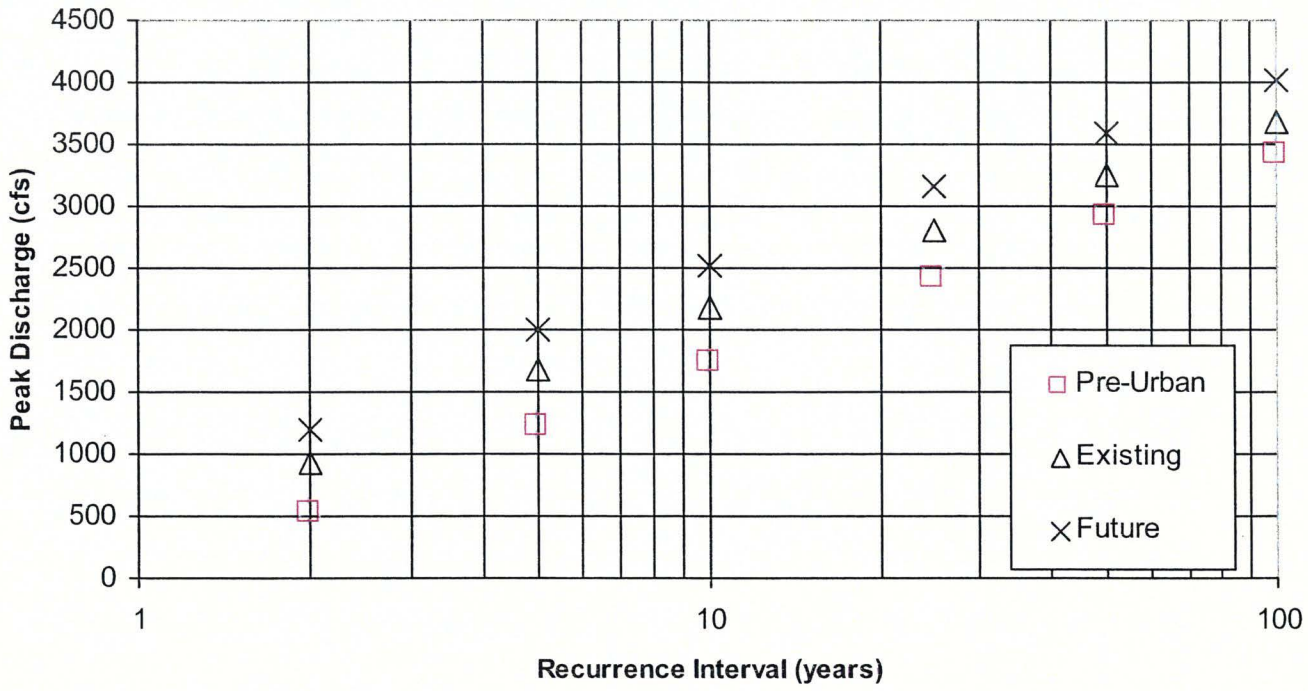
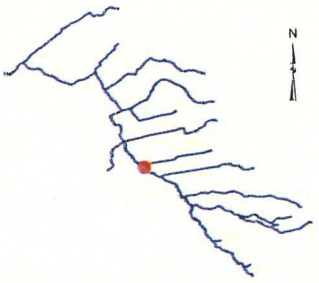


Figure 4-24
Thompson Creek Flood Frequency for Junction J-11 Using Event-Based Model Results



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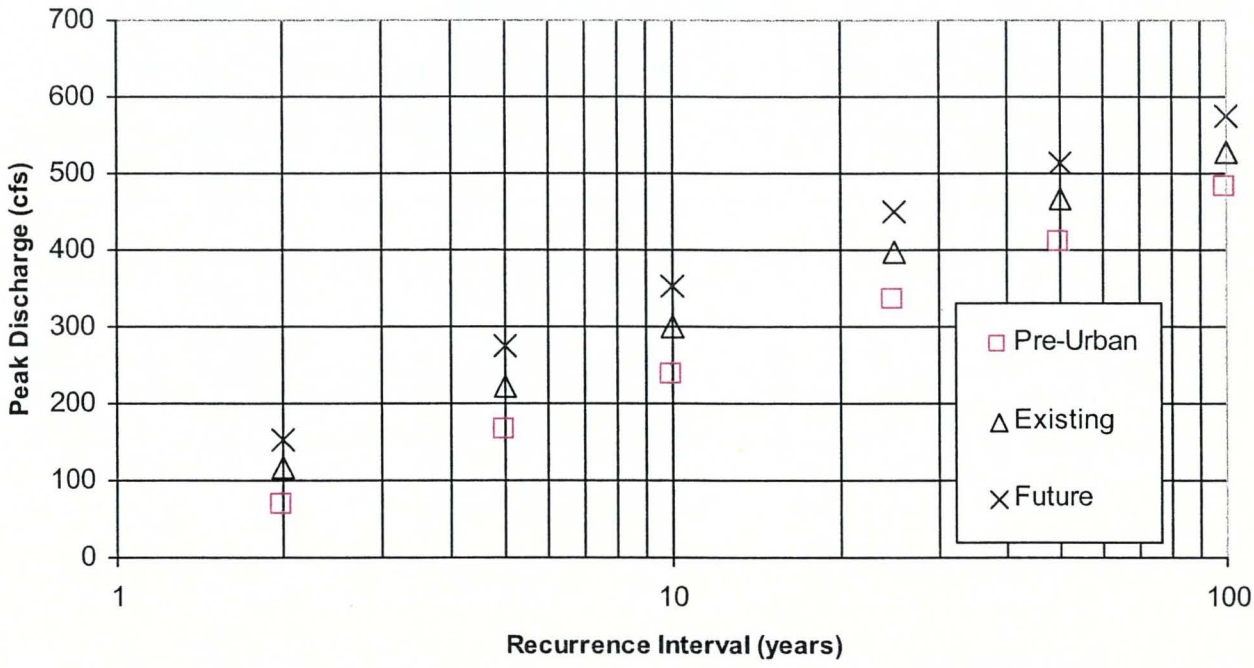
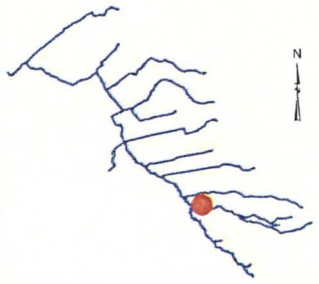


Figure 4-25
Thompson Creek Flood Frequency
at Cross-Section YB-7
Using Event-Based Model Results
(Results are inclusive of flows upstream of YB-7 on Yerba Buena)



Cross-section Location

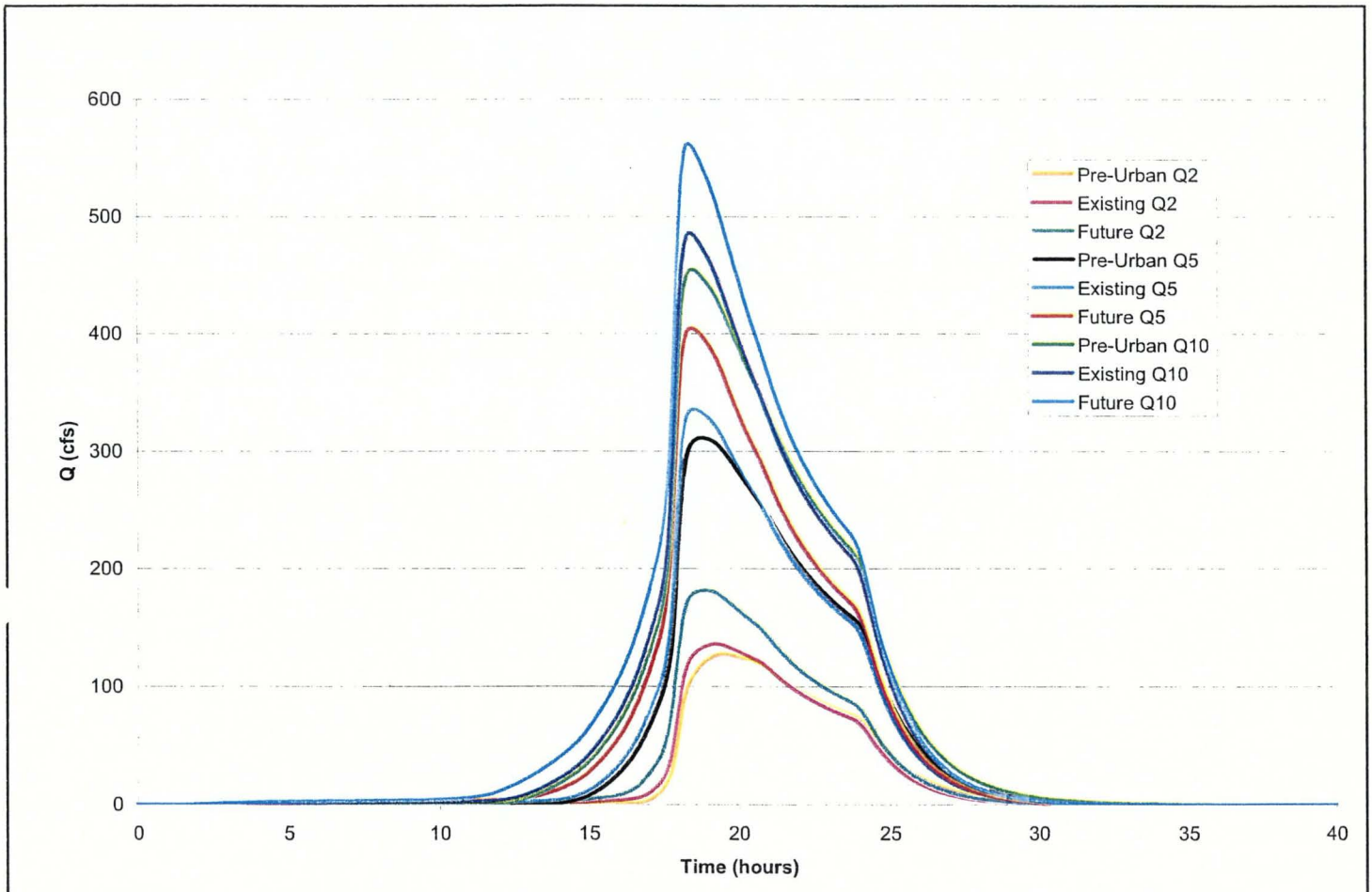


Figure 4-26
Thompson Creek Event-Based Storm Hydrographs at Junction J-3 for a 2-yr, 5-yr and 10-yr Event Using Pre-Urban, Existing and Future Land Use Conditions

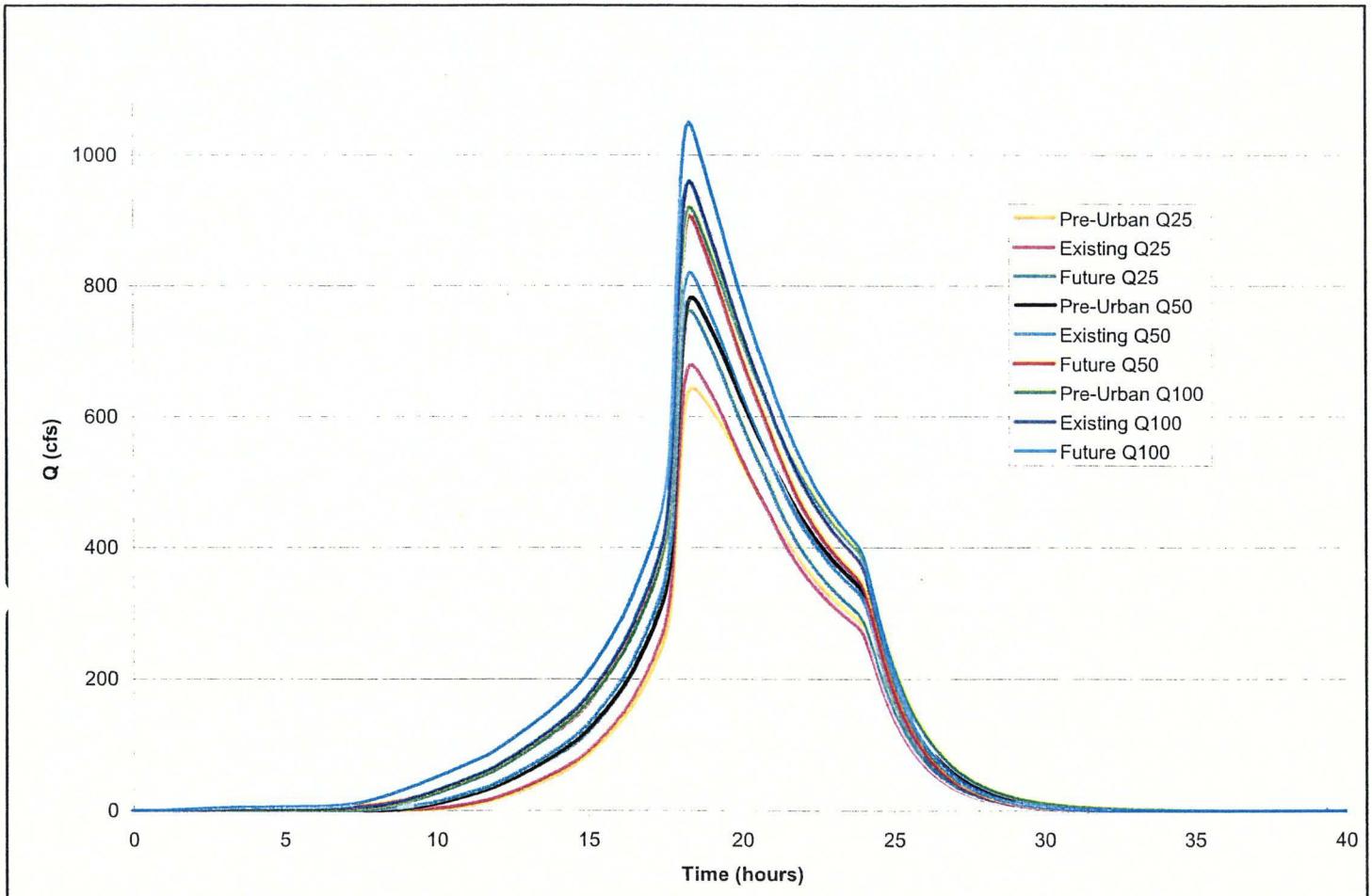


Figure 4-27
Thompson Creek Event-Based Storm Hydrographs at Junction J-3 for a 25-yr, 50-yr and 100-yr Event Using Pre-Urban, Existing and Future Land Use Conditions

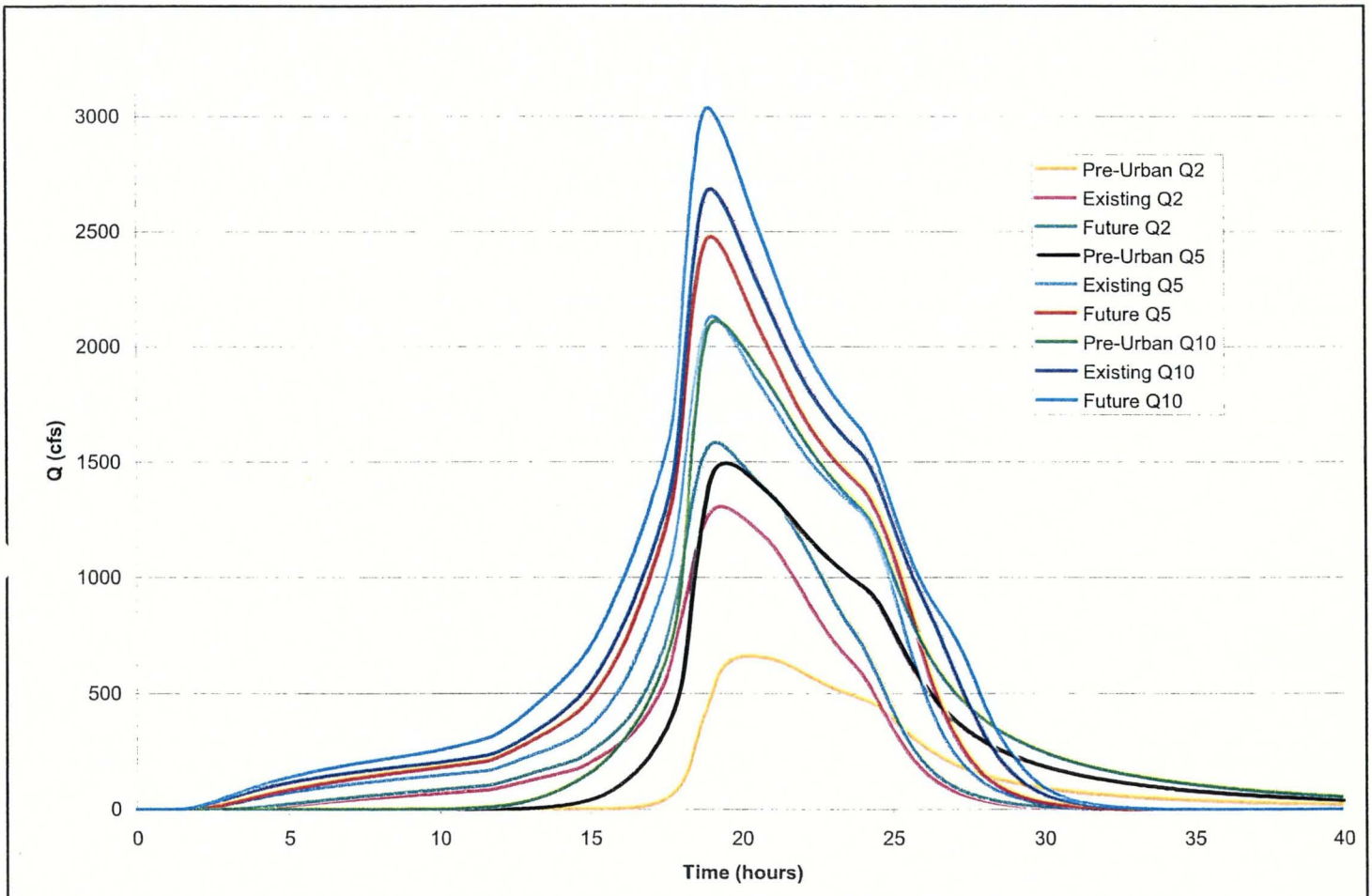


Figure 4-28
Thompson Creek Event-Based Storm Hydrographs at Junction J-12 for a 2-yr, 5-yr and 10-yr Event Using Pre-Urban, Existing and Future Land Use Conditions

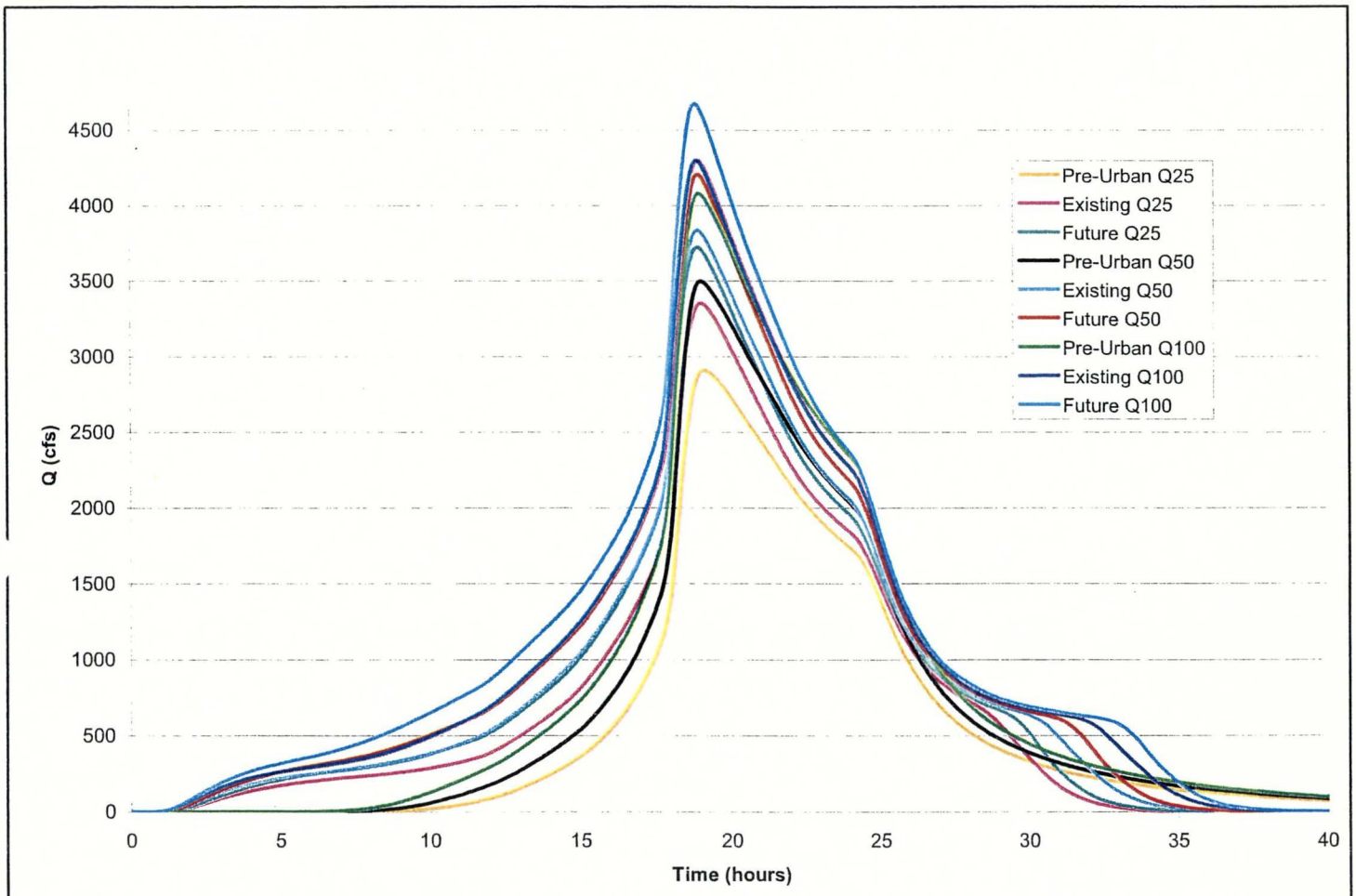


Figure 4-29
Thompson Creek Storm Hydrographs at Junction J-12 for a 25-yr, 50-yr and 100-yr Event Using Pre-Urban, Existing and Future Land Use Conditions

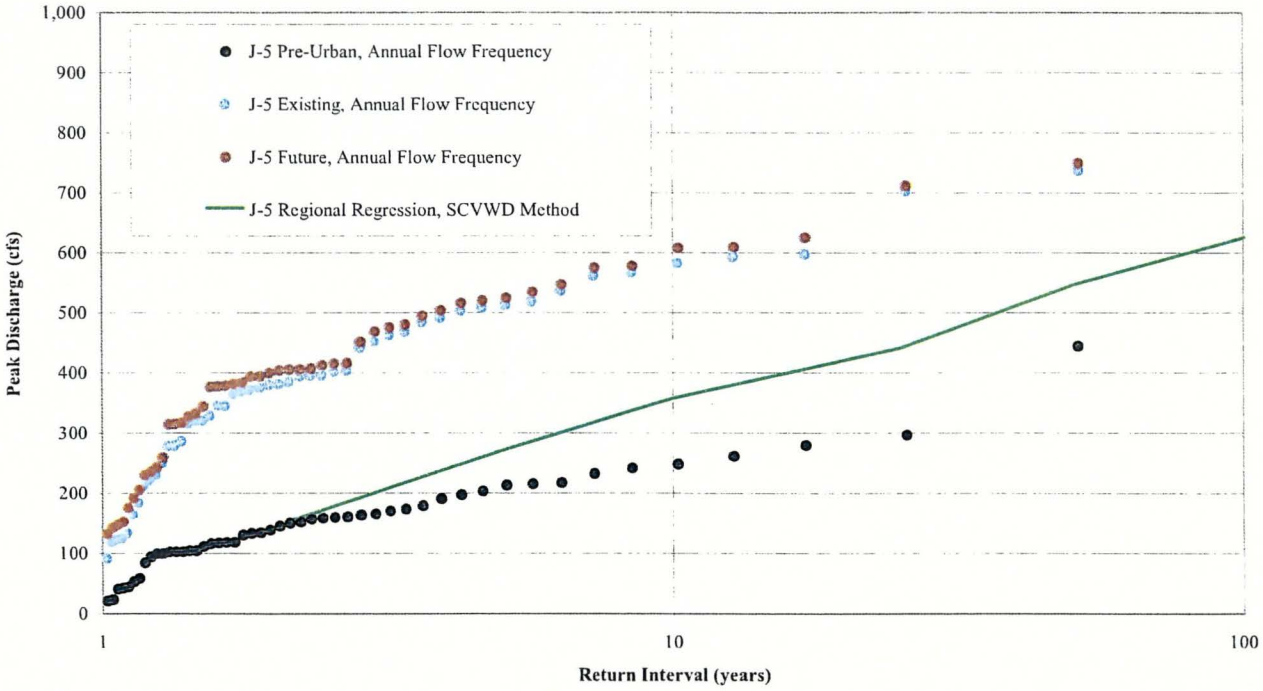
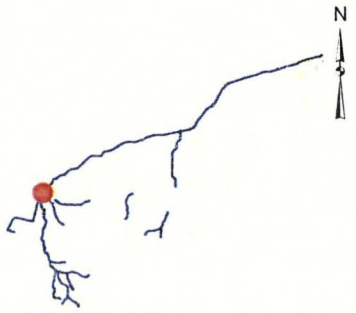


Figure 4-30
Flood Frequency for Ross Creek Junction J-5
Using Continuous Model Results



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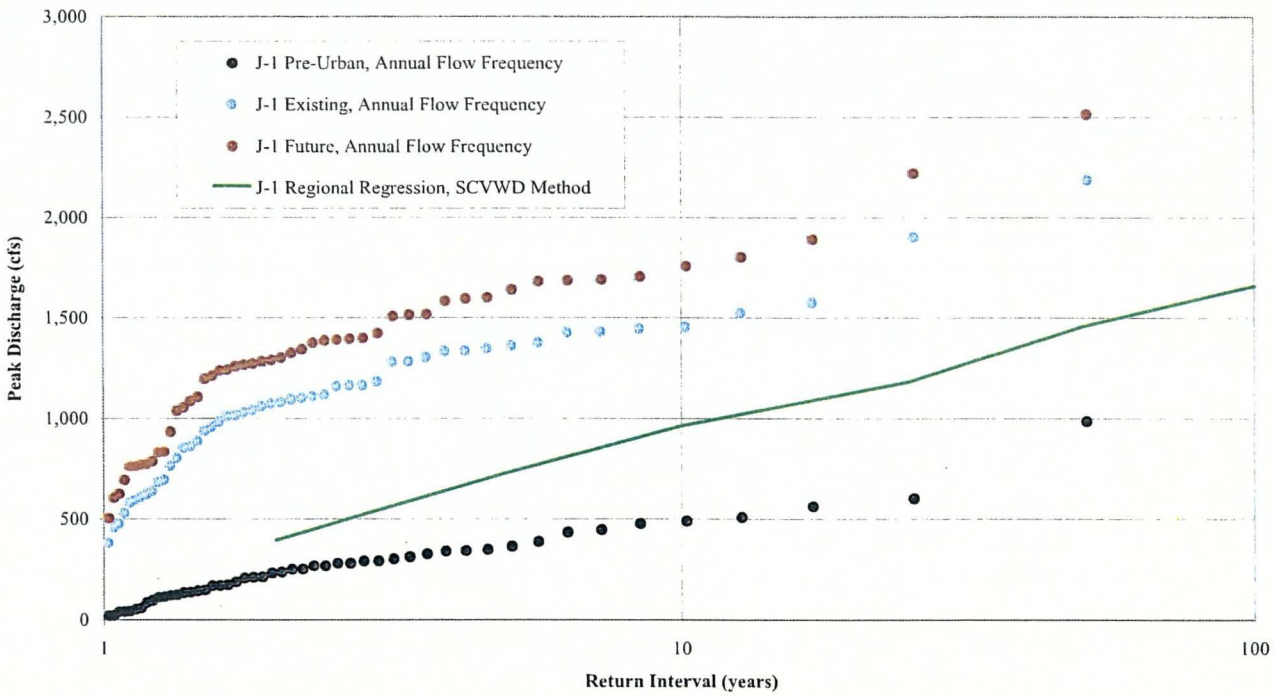
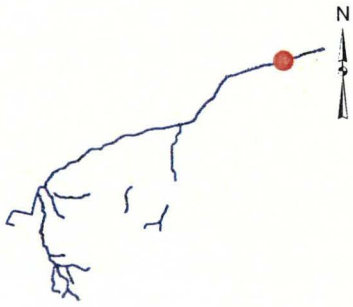


Figure 4-31
Flood Frequency for Ross Creek Junction J-1
Using Continuous Model Results



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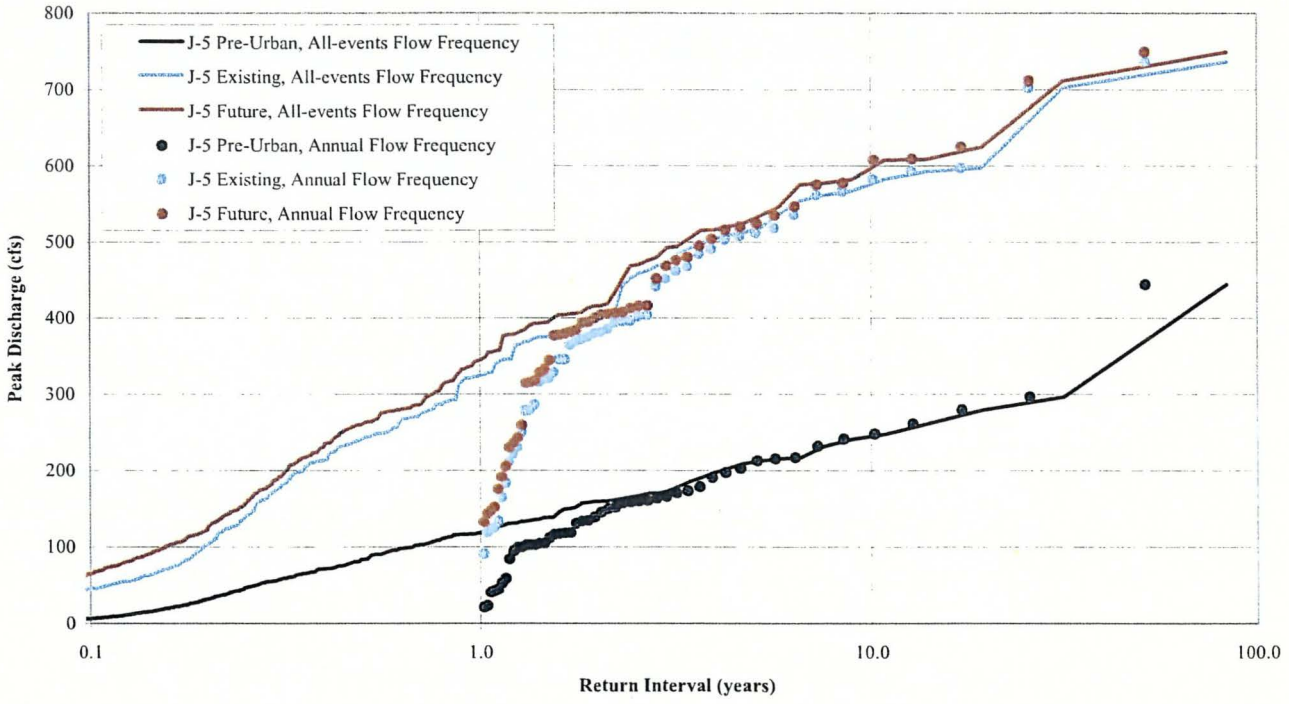
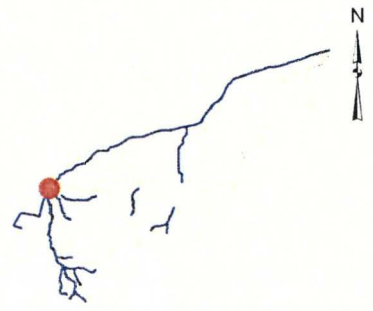


Figure 4-32
Ross Creek Peak Flow Frequency Analysis: Monthly and Annual Recurrence Intervals at Junction J-5



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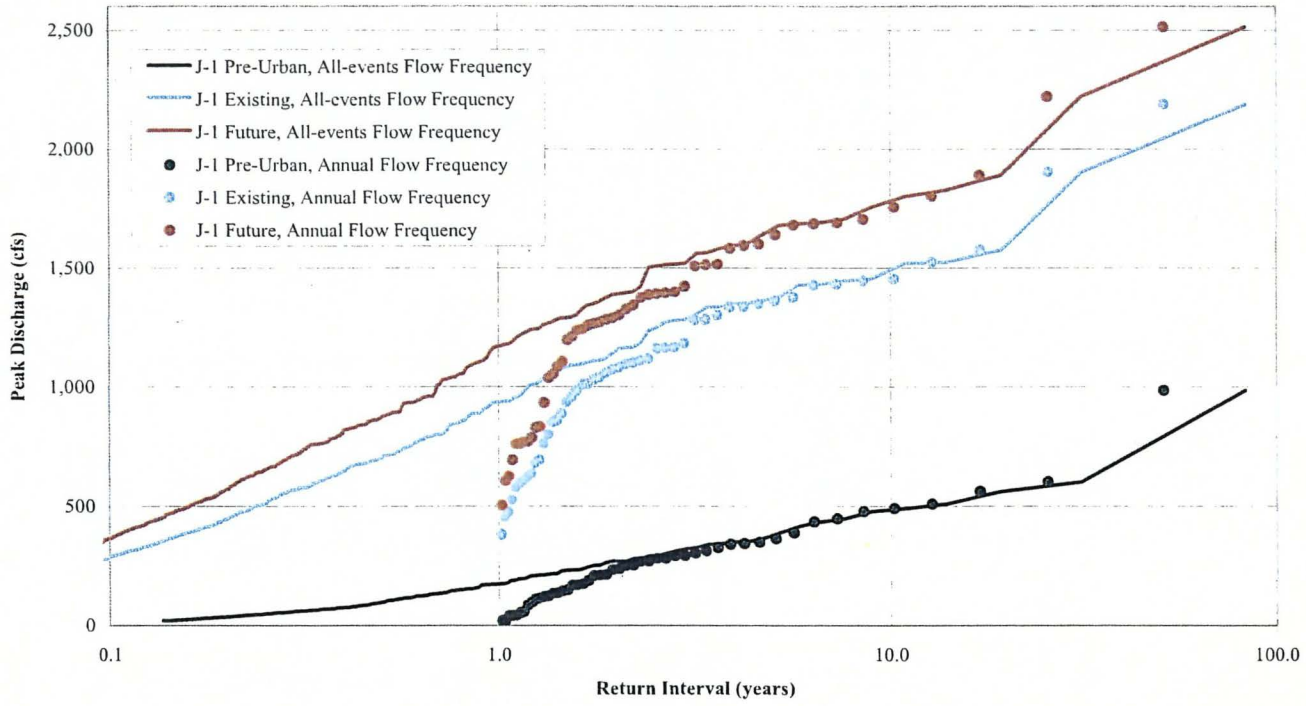
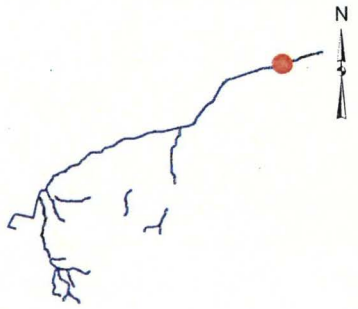


Figure 4-33
Ross Creek Peak Flow Frequency Analysis: Monthly and Annual Recurrence Intervals at Location J-1



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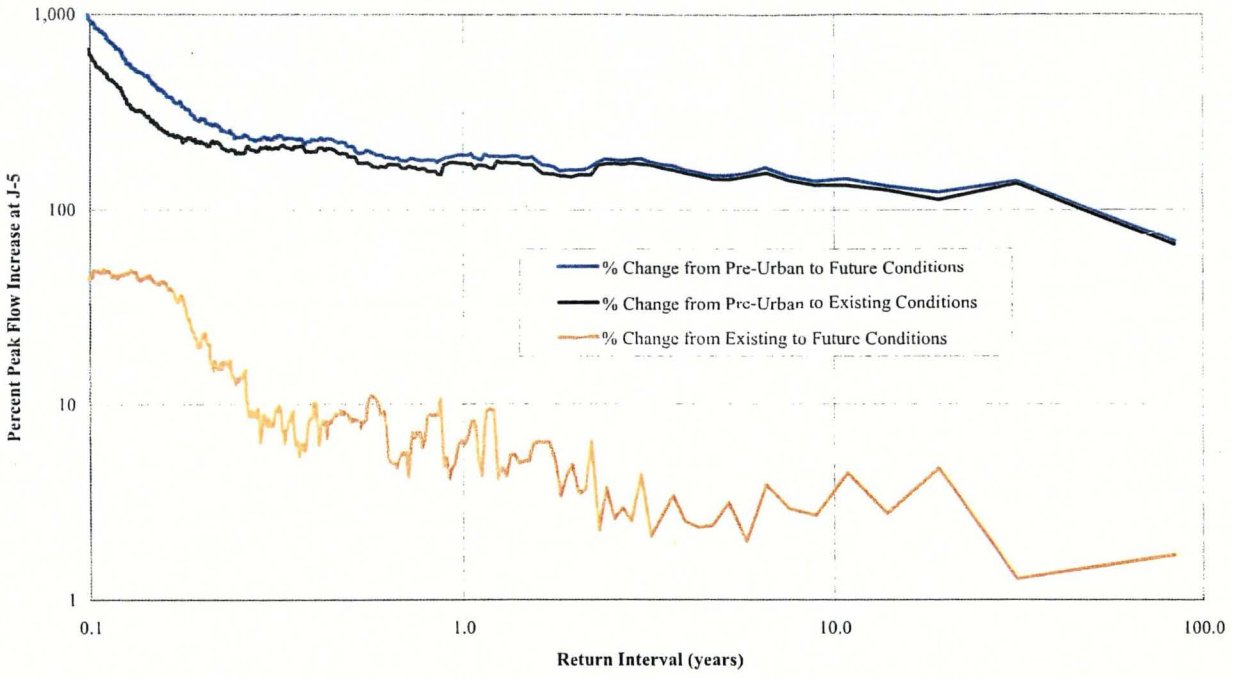
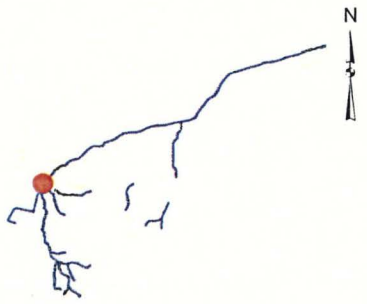


Figure 4-34
Percent Flow Increase at Ross Creek at Junction J-5



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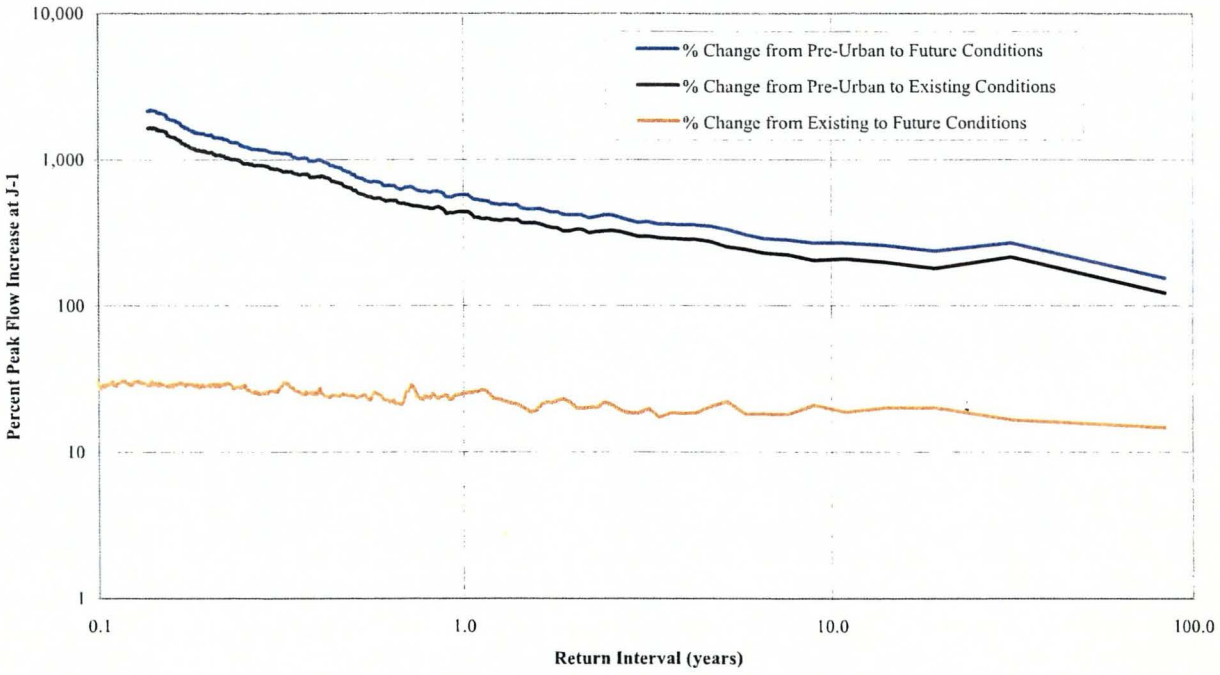
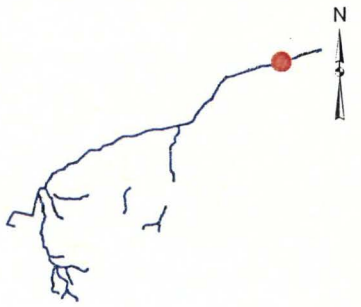


Figure 4-35
Percent Flow Increase at Ross Creek Junction J-1



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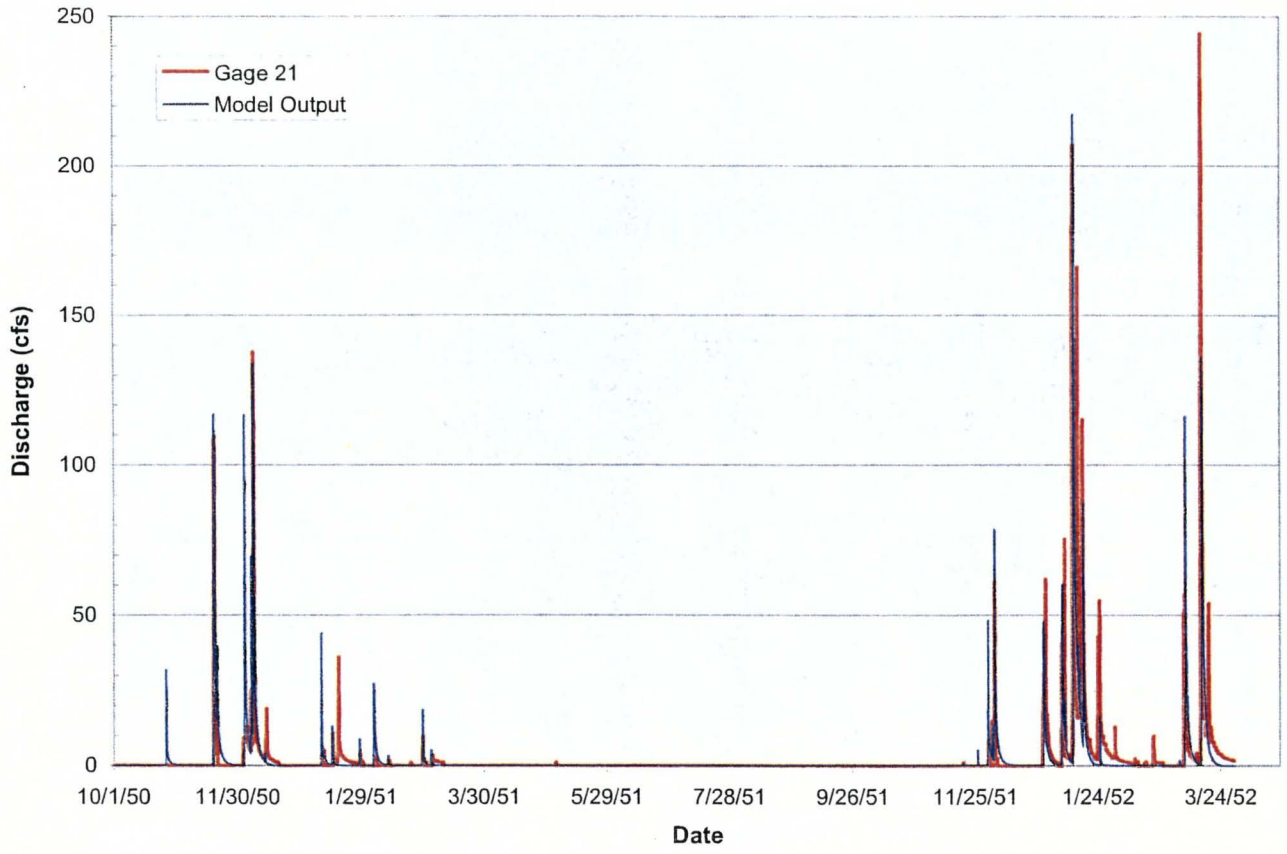


Figure 4-36
Pre-Development Calibration Hydrographs for Gage 21 and Model Junction J-5

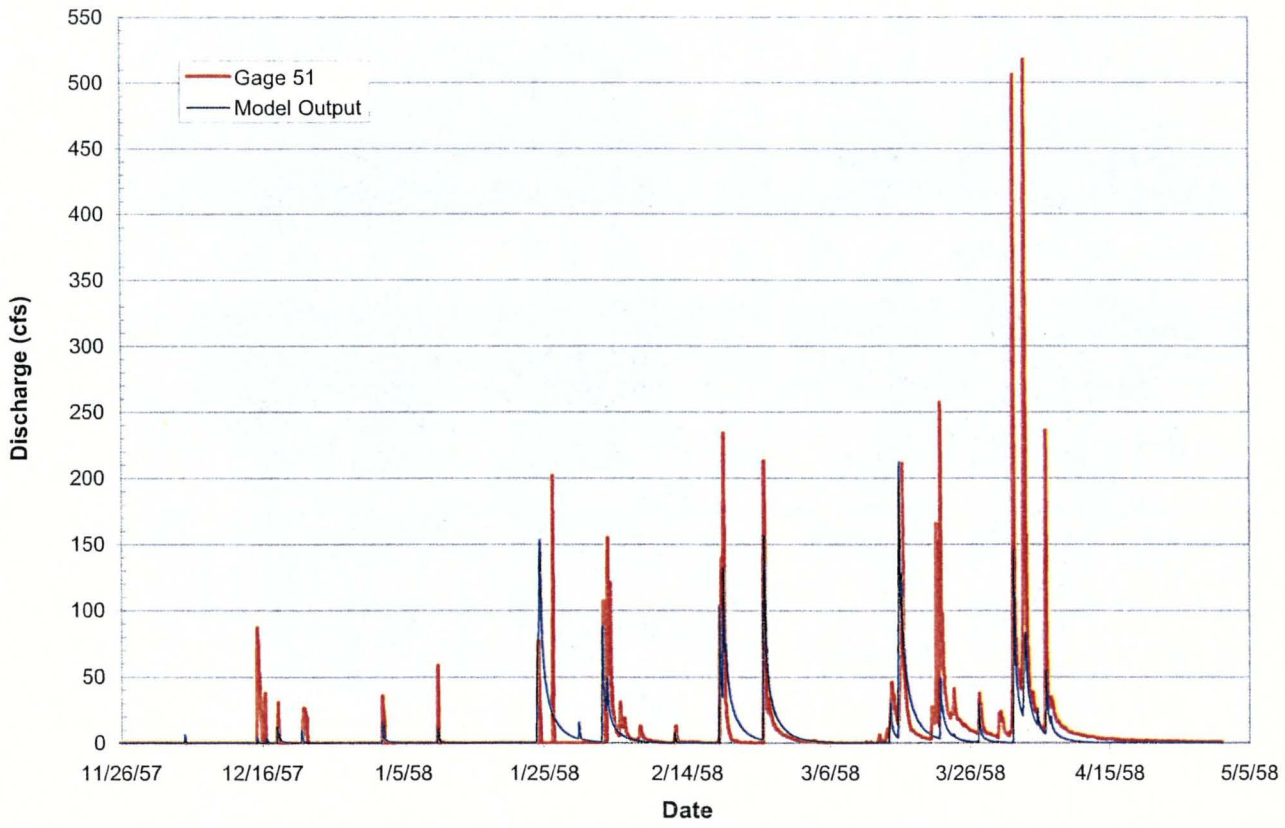


Figure 4-37
Pre-Development Calibration Hydrographs for Gage 51 and Model Junction J-1

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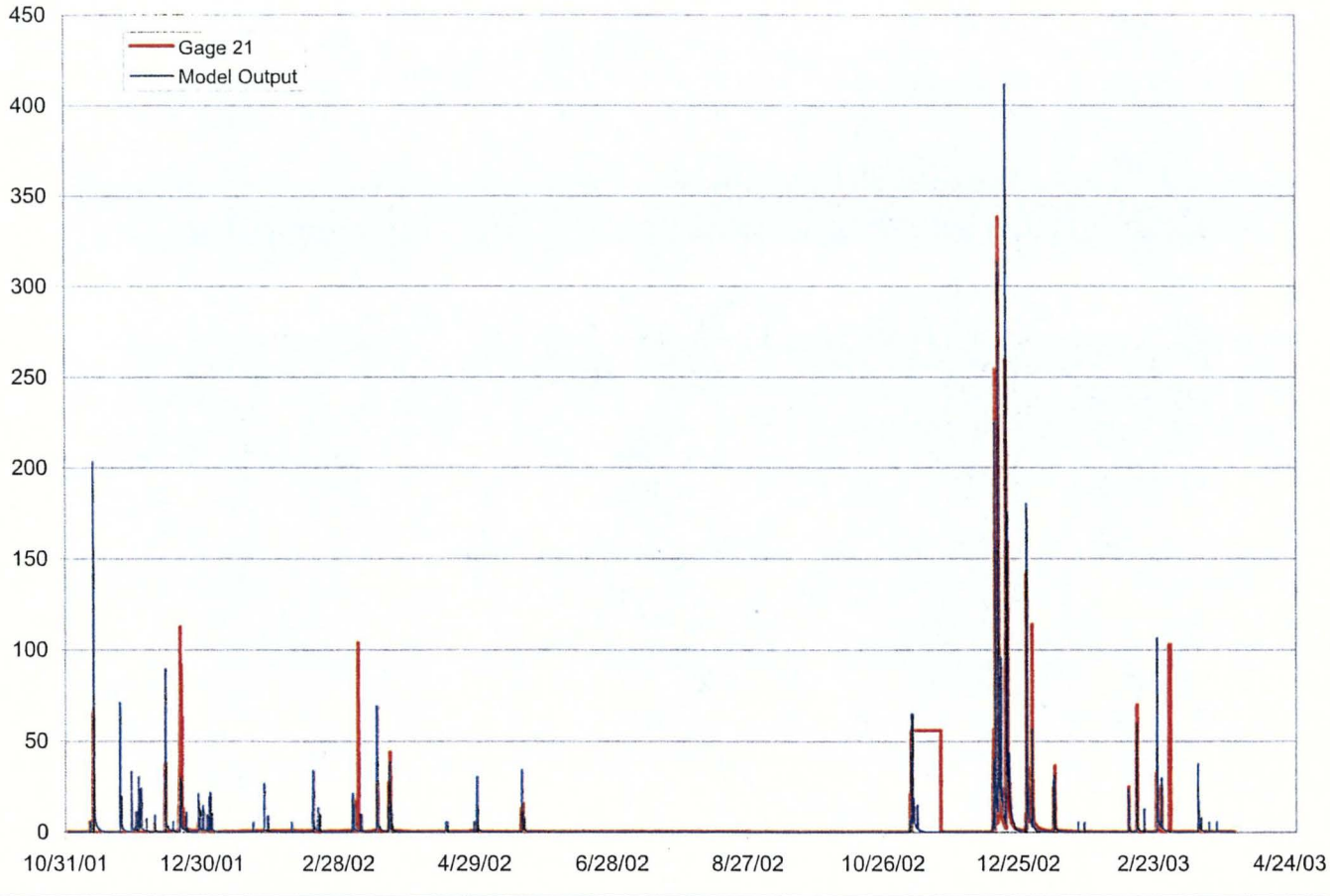


Figure 4-38
Existing Condition Calibration Hydrographs for Gage 21 and Model Junction J-5

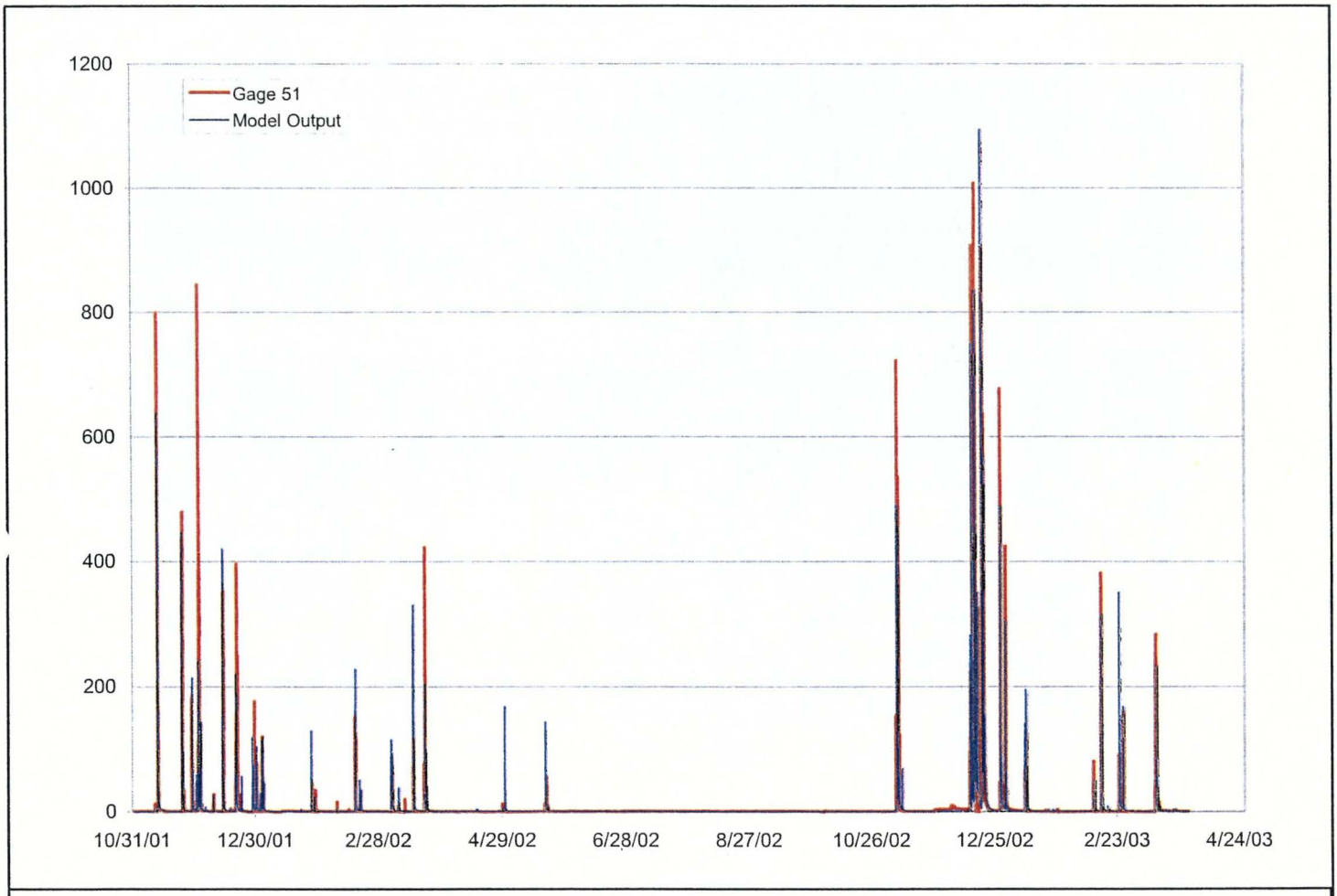
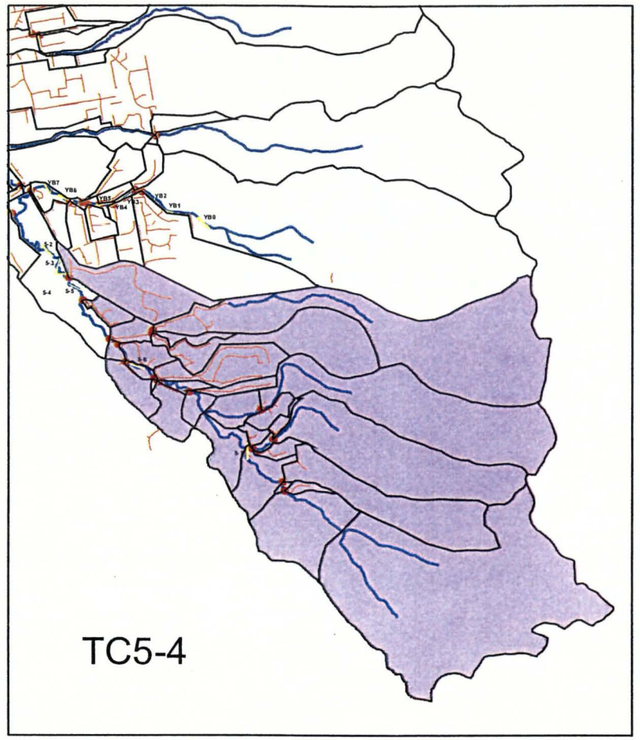
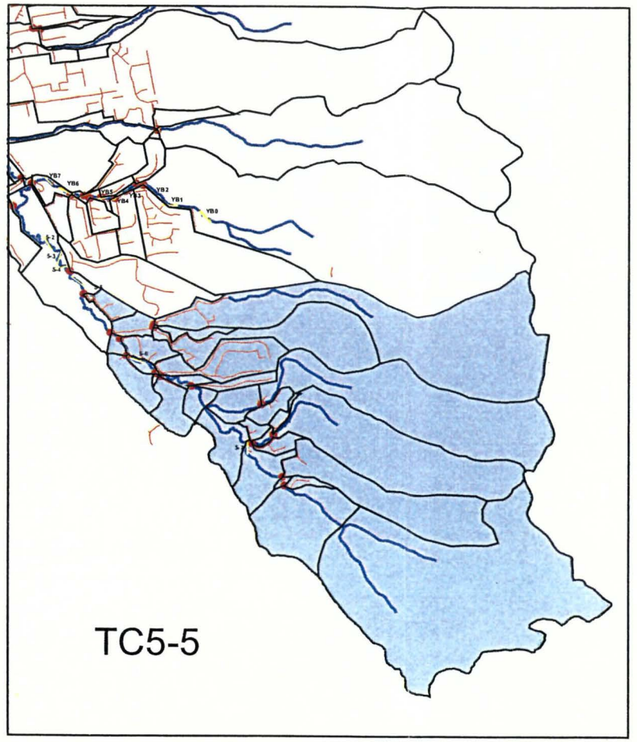


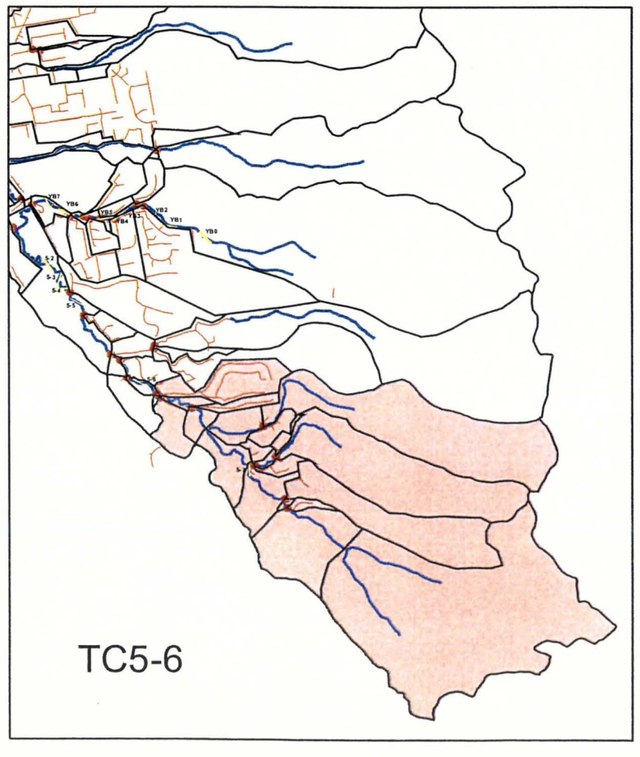
Figure 4-39
Existing Condition Calibration Hydrographs for Gage 51 and Model Junction J-1

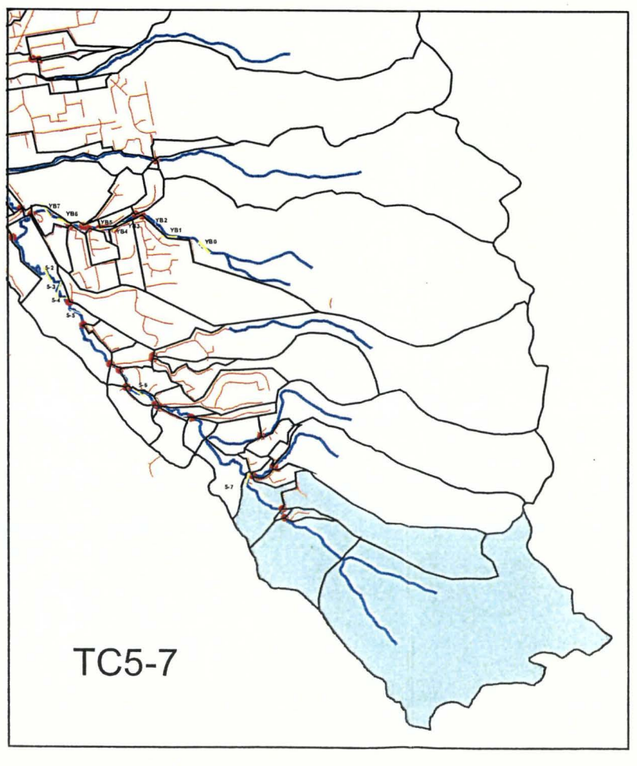
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X-section	Area acre	Area IMP acre	Projected Area FIMP acre	% IMP	Projected % FIMP
TC5-4	3,918	434	885	11.1	22.6
TC5-5	3,758	355	783	9.4	20.8
TC5-6	2,736	188	471	6.9	17.2
TC5-7	1,504	23	169	1.6	11.3





Feet

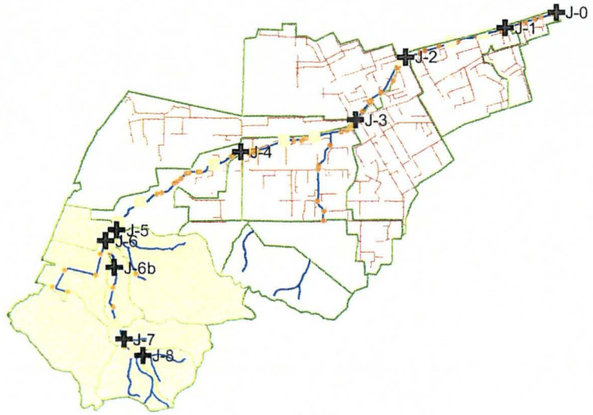
0 5,500 11,000

N

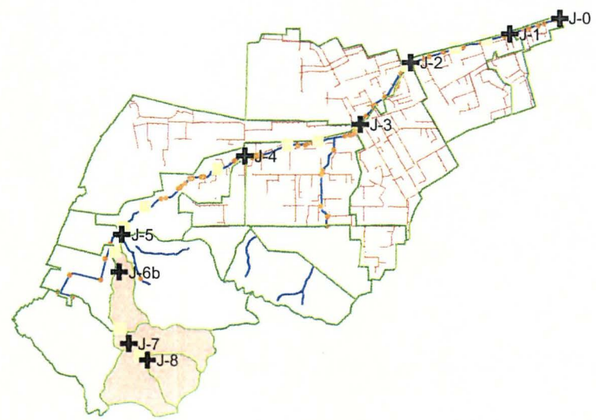
Figure 5-1T
Areas and Percent Imperviousness for Sub-catchments Draining into Cross-sections and Junctions in Thompson Creek

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J-5



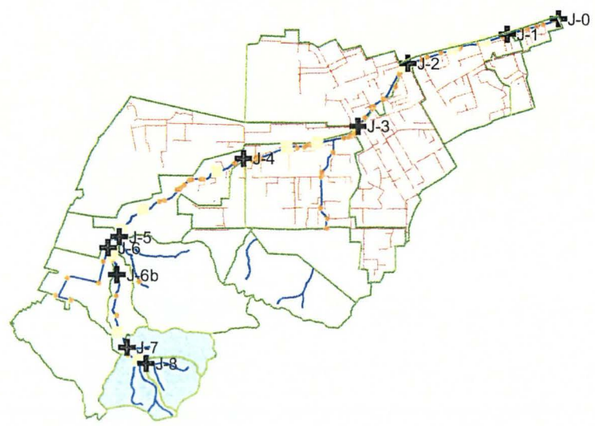
J-6b



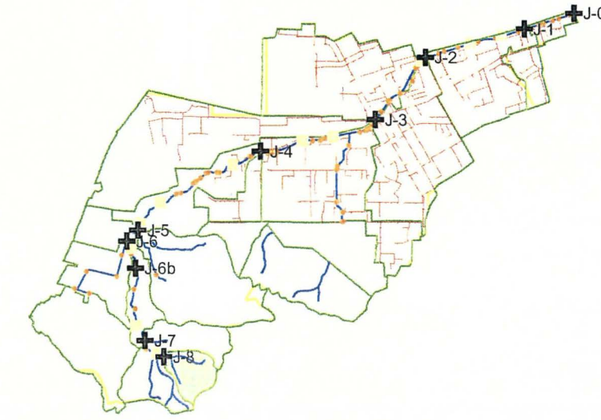
Cross-Section Name	Area*	Area IMP	Projected Area IMP	% IMP	Projected % IMP
J-5	1764.8	329.4792	631.0654	18.7	35.8
J-6b	540.1	48.1426	144.7229	8.9	26.8
J-7	417.6	24.3776	97.5604	5.8	23.4
J-8	115	1.38	20.7	1.2	18.0

* areas measured in acres

J-7



J-8



Legend

- Junction
- Cross-Section
- Outfall
- Catchment Boundary
- Creek
- Mains & Laterals

N

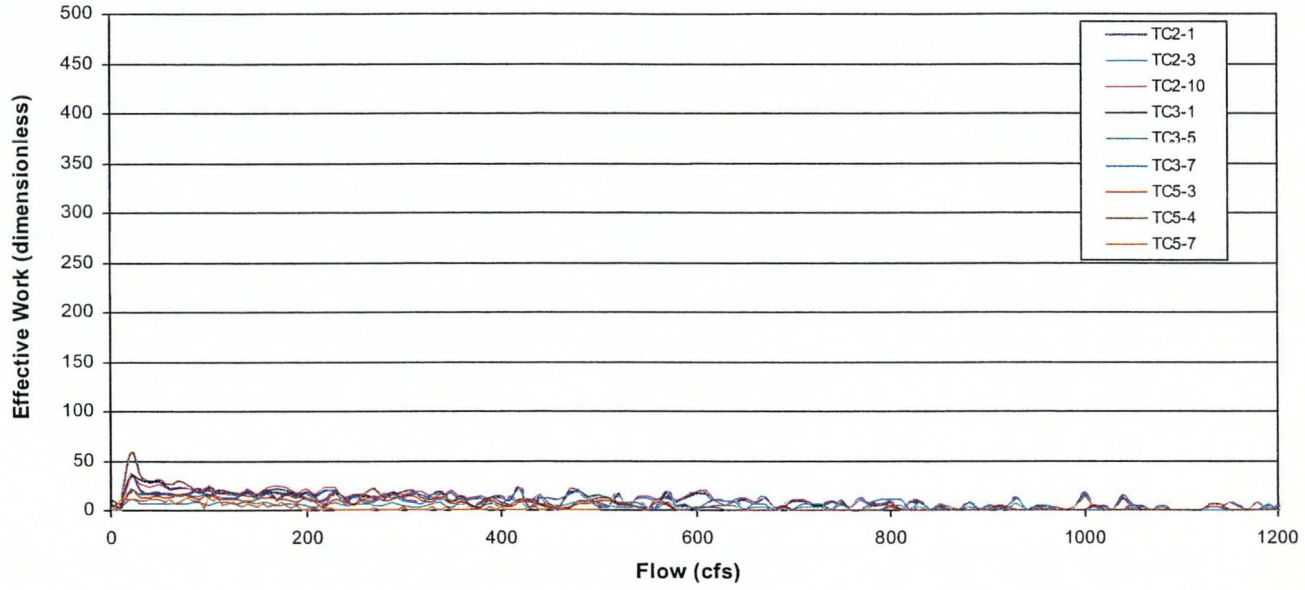
Feet
0 7,500

Figure 5-1R
Areas and Percent Imperviousness for Sub-catchments Draining into Cross-Sections and Junctions in Ross Creek Junctions

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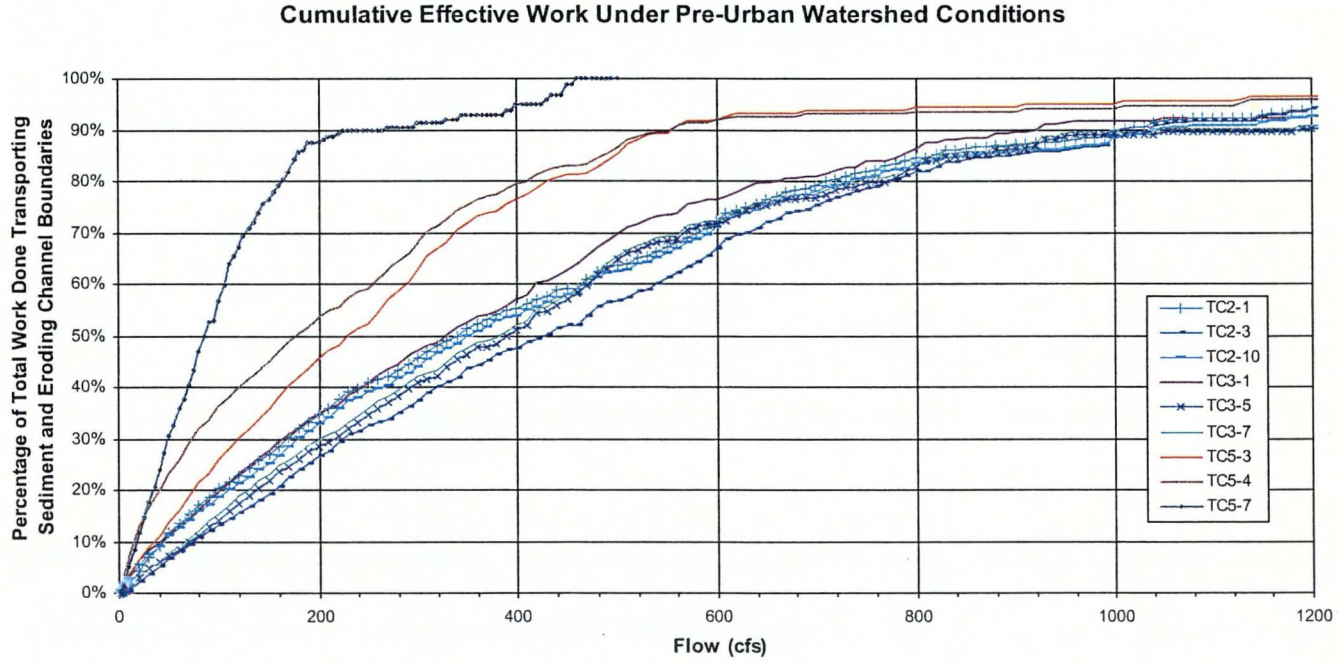
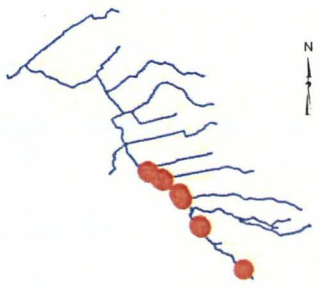


Figure 5-2T
Effective Work Curves for Pre-urban Watershed Conditions for Thompson Creek



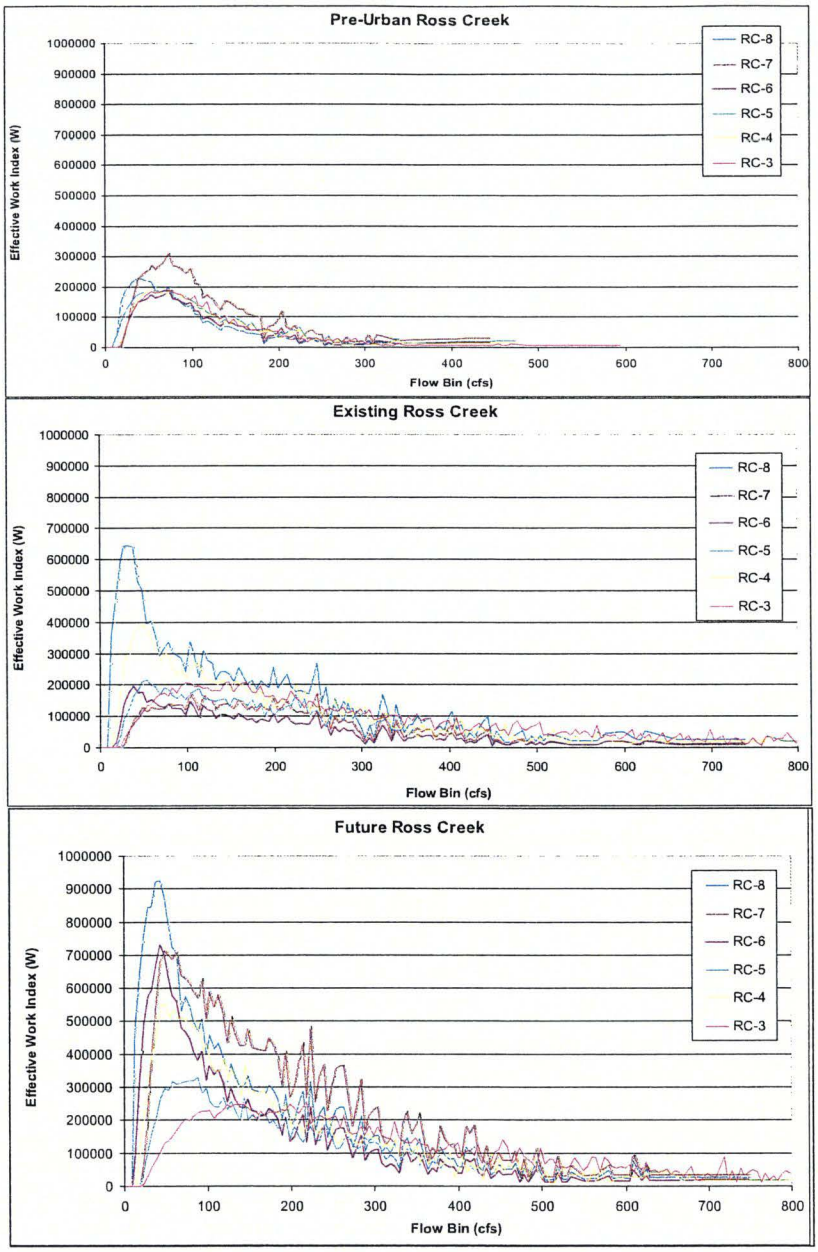
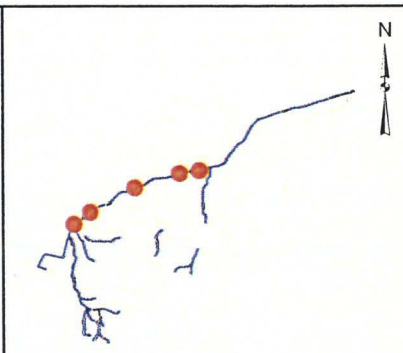


Figure 5-2A
Effective Work Curves for Pre-urban, Existing, and Future Watershed Conditions for Ross Creek



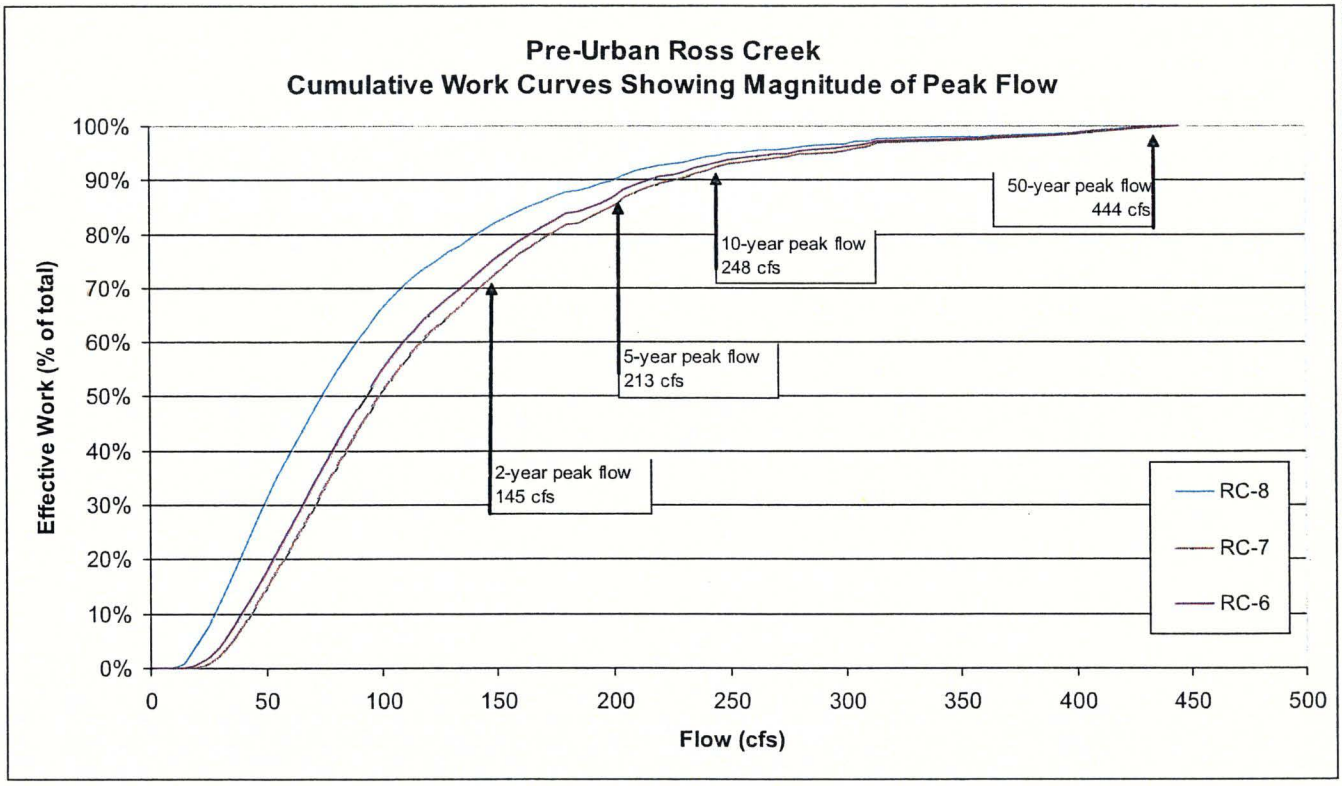
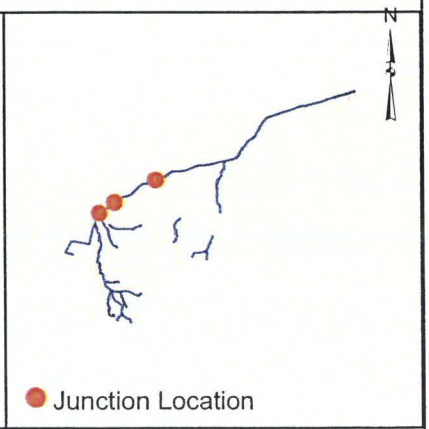


Figure 5-2B
Cumulative Work Curves for Pre-urban, Existing and Future Watershed Conditions for Ross Creek



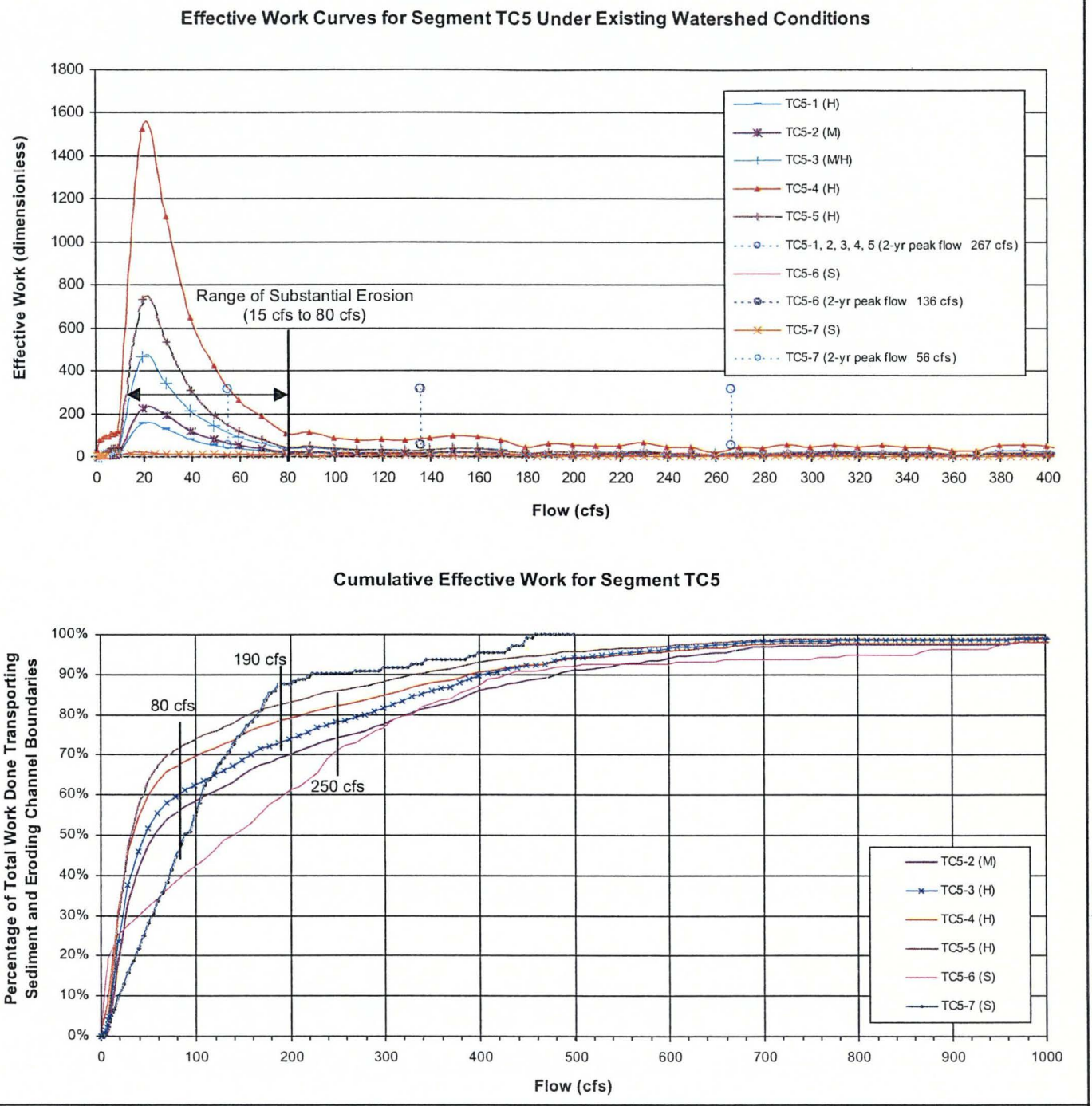
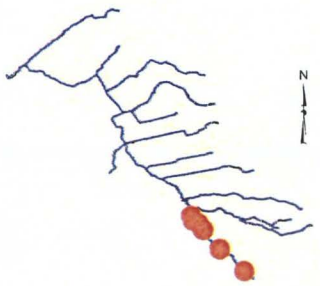


Figure 5-3T
Effective Work Curves for Existing Watershed Conditions for Thompson Creek



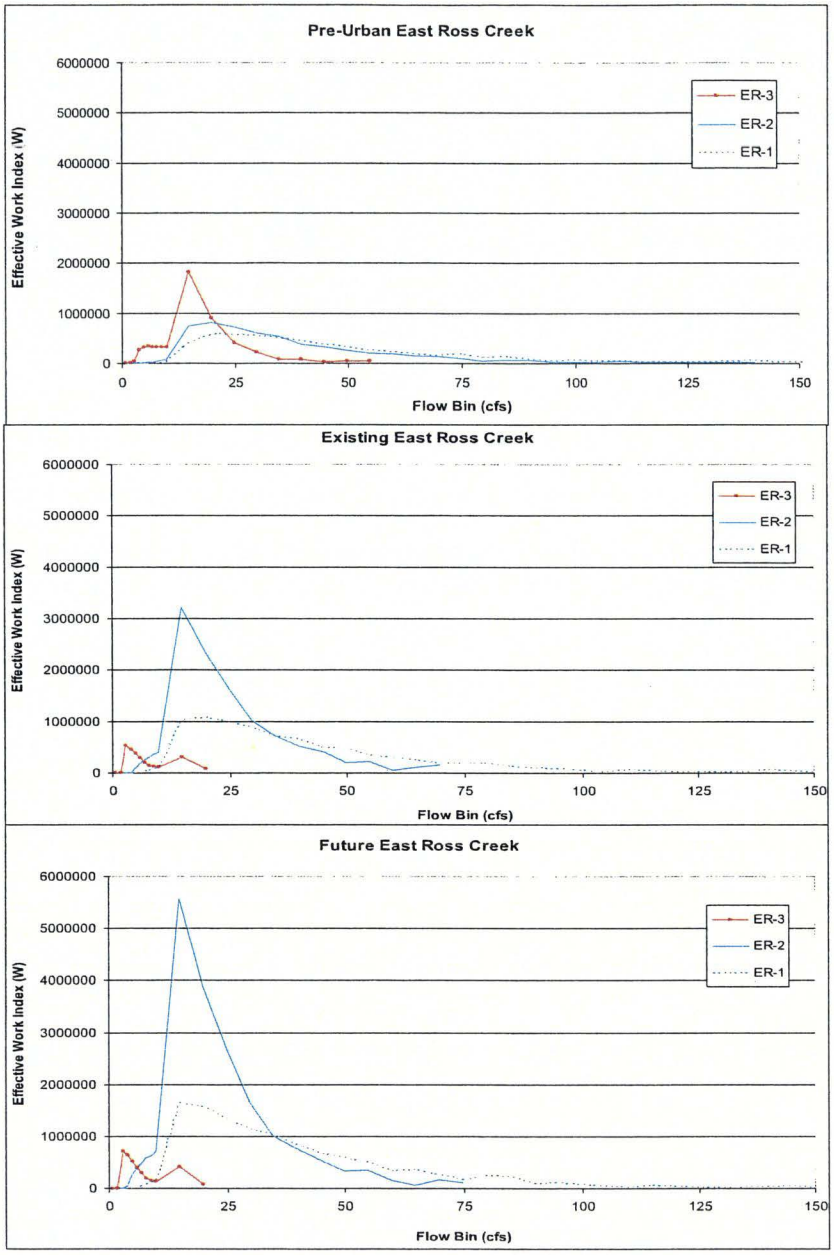
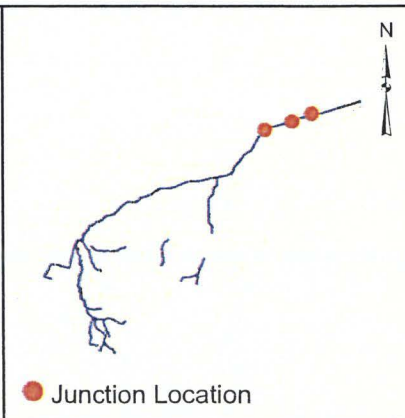


Figure 5-3
Effective Work Curves for Pre-urban, Existing, and Future Watershed Conditions for East Ross Creek

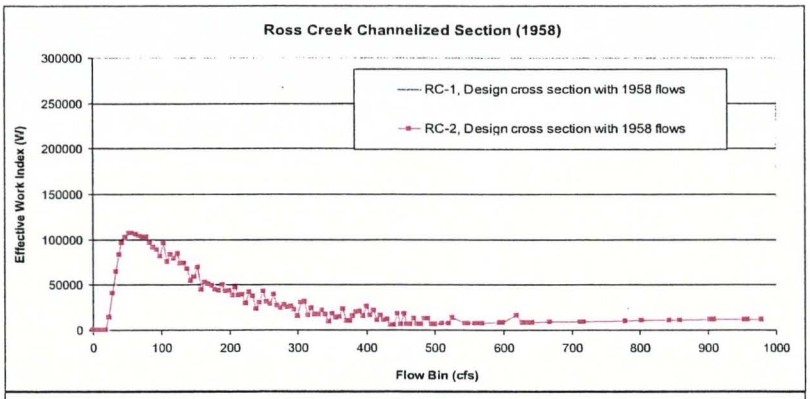


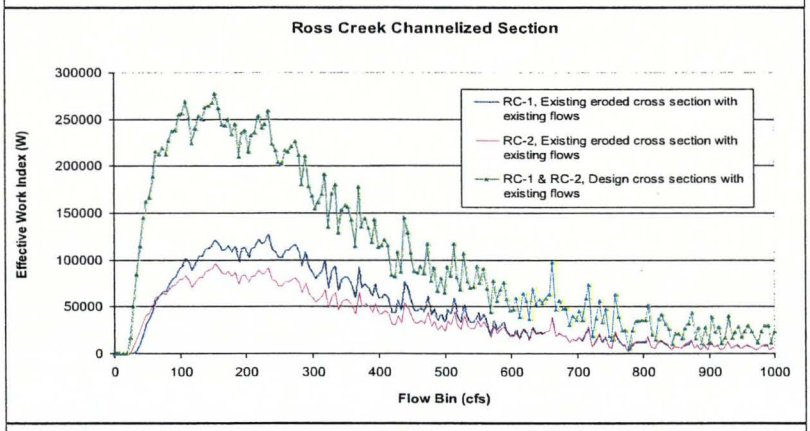
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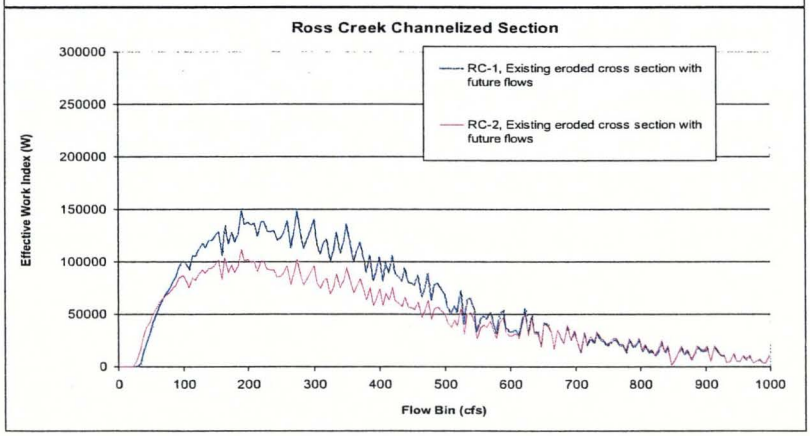
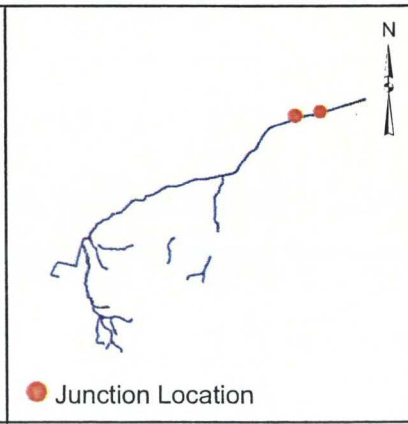


Figure 5-4
Effective Work Curves for Pre-urban, Existing, and Future Watershed Conditions for RC-1 and RC-2: Channelized Sections



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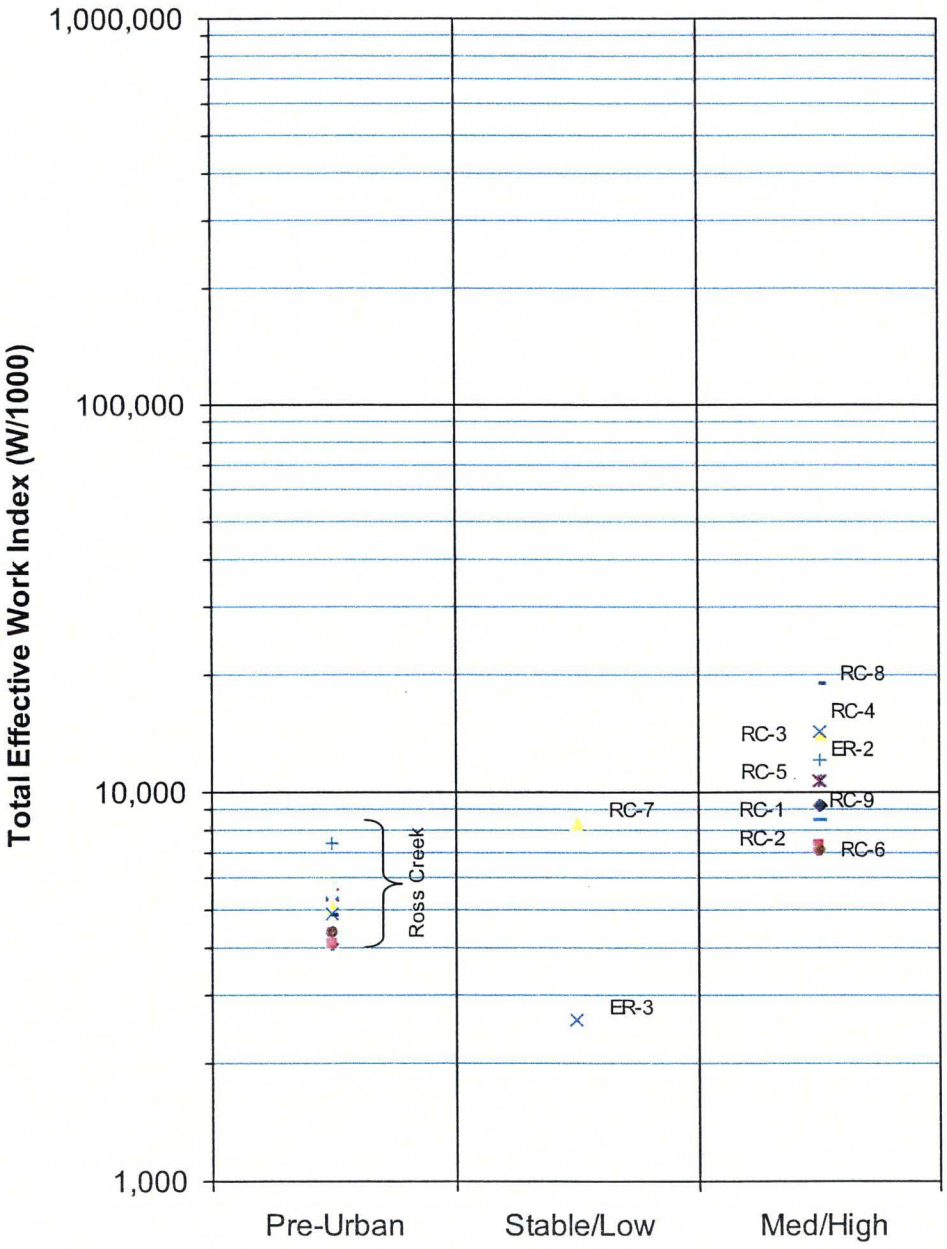


Figure 5-5
Total Effective Work Index for Ross Creek: Continuous Modeling Using Existing and Pre-Urban Flows

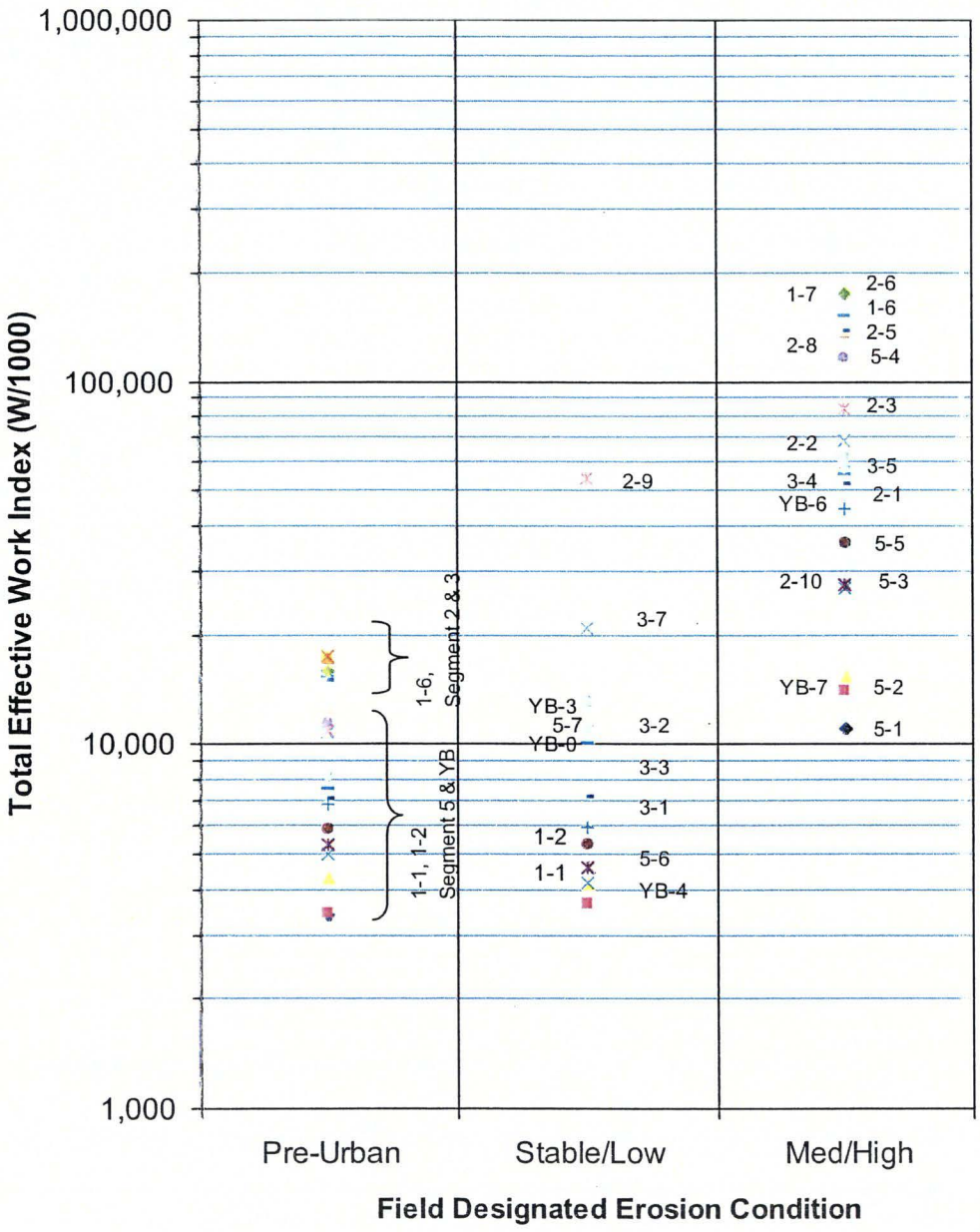


Figure 5-6
Total Effective Work Index for Thompson Creek and Yerba Buena Creek: Continuous Modeling Using Existing and Pre-Urban Flows

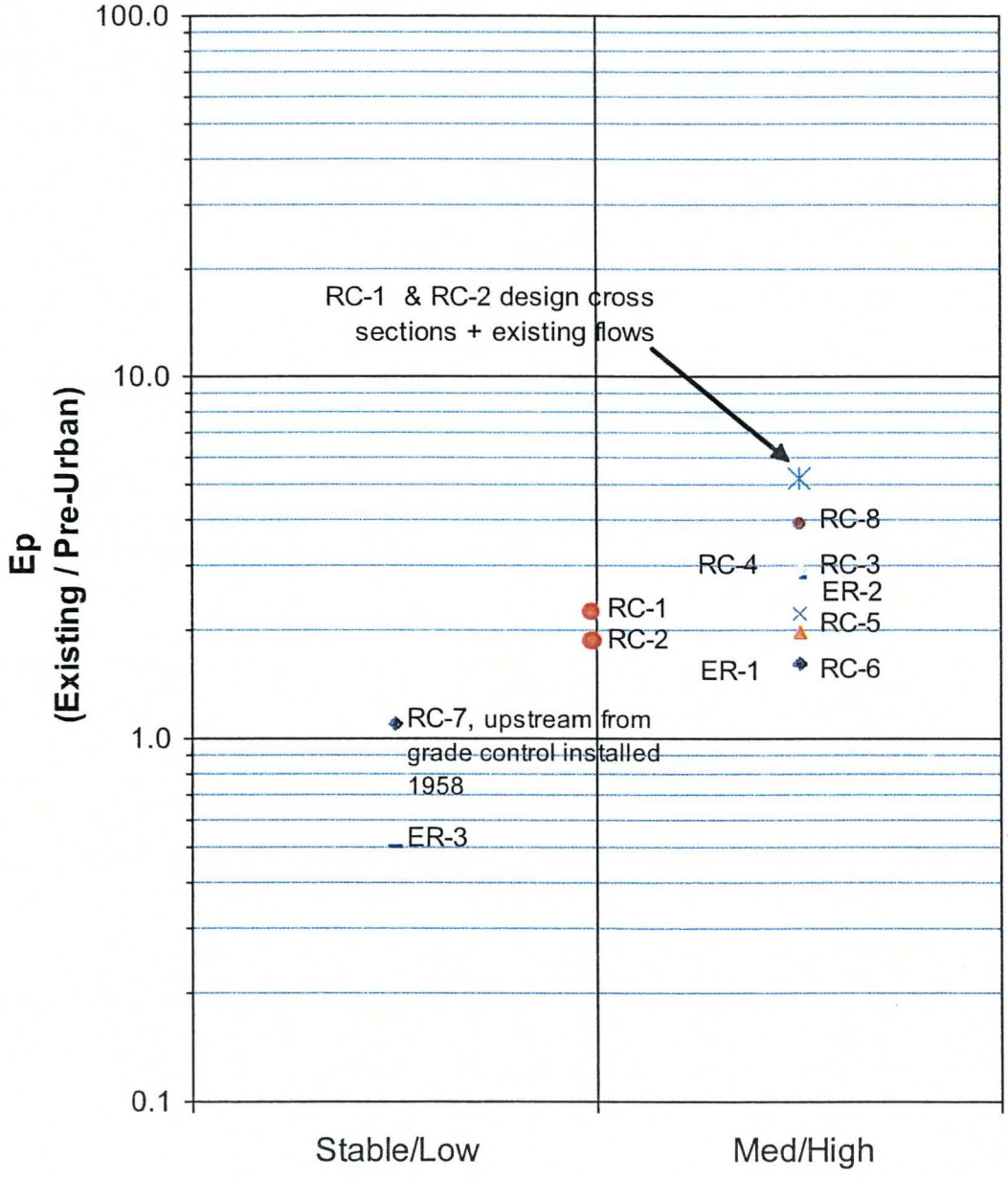


Figure 5-7
Erosion Potential Chart for Ross Creek:
Continuous Modeling Using Existing Flows

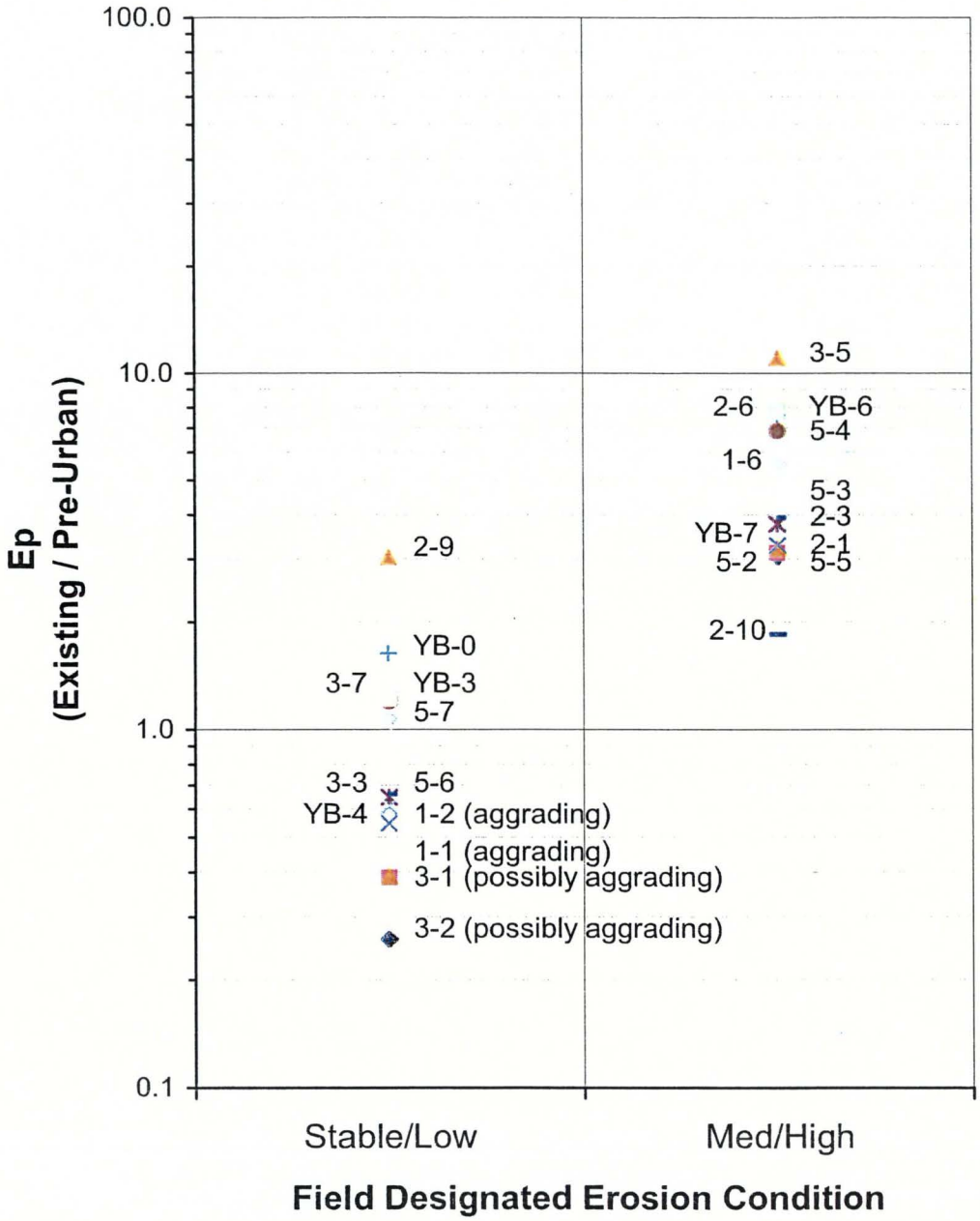


Figure 5-8
Erosion Potential Chart for Thompson Creek:
Continuous Modeling Using Existing Flows

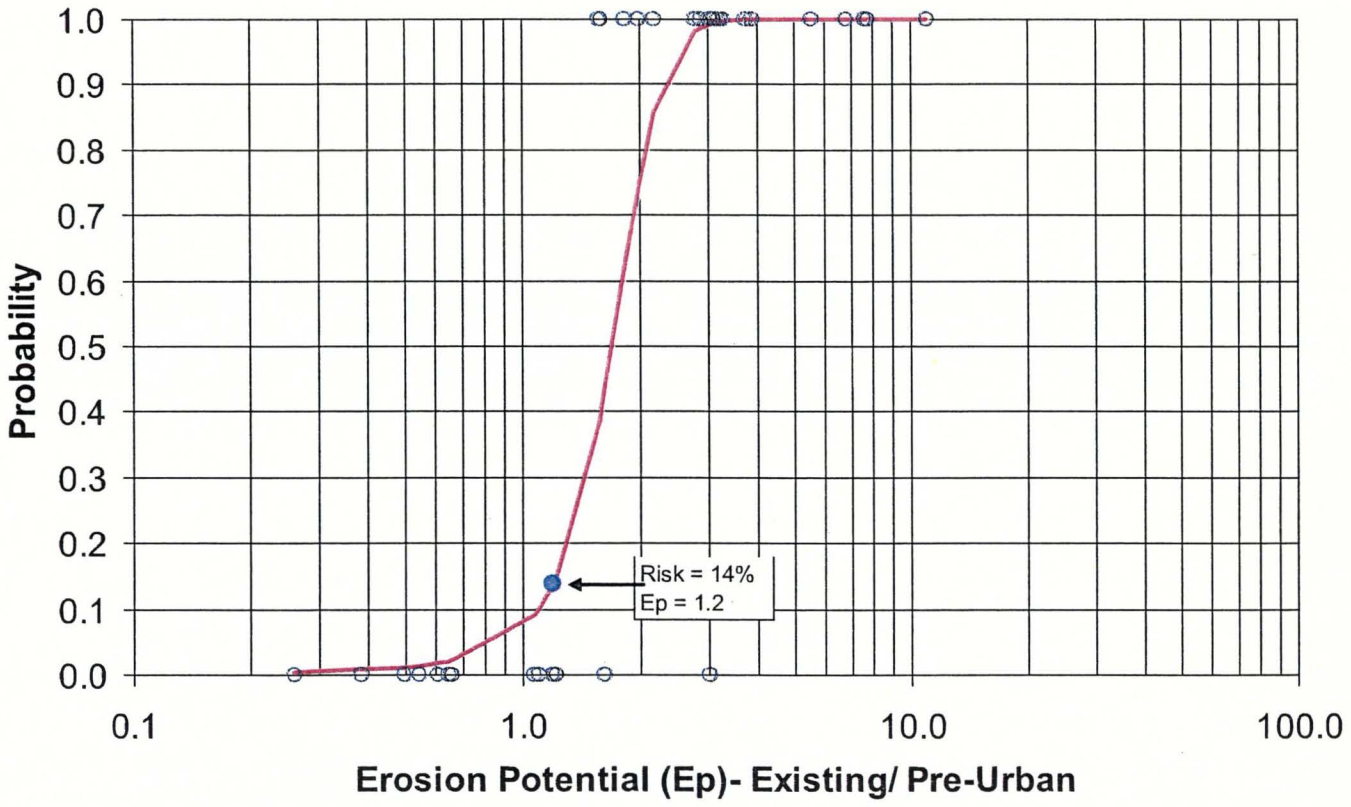


Figure 5-9
Probability of Erosion Instabilities for Ross Creek

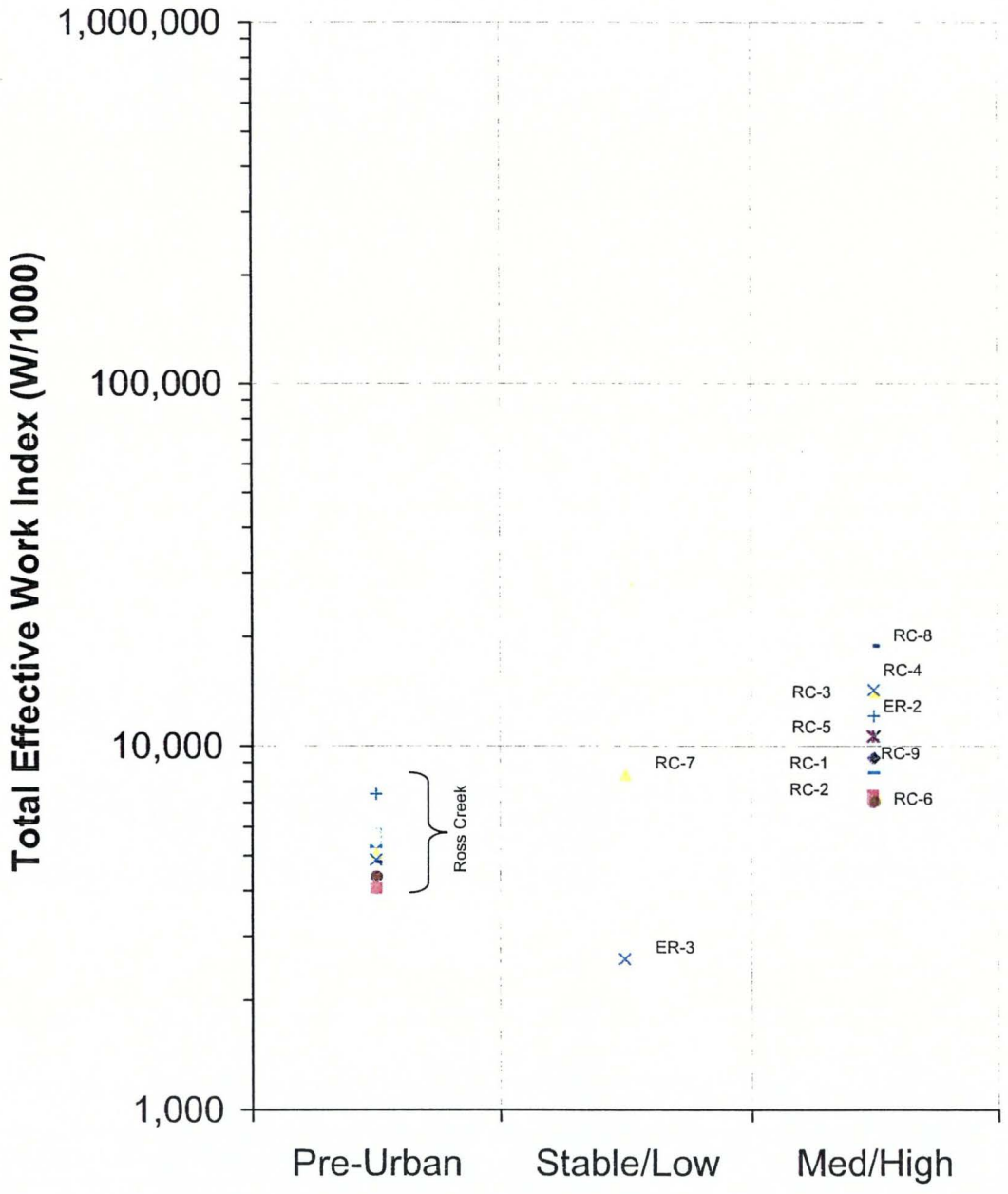


Figure 5-10
Erosion Potential Chart for Ross Creek:
Continuous Modeling Using Future Flows

Legend

- GPS Point
- Major Creek
- Minor Creek
- HMP Catchment Boundary
- Adobe Creek Catchment
- Alamitos Creek Catchment
- Baron Creek Catchment
- Berryessa Creek Catchment
- Calabazas Creek Catchment
- Calera Creek Catchment
- Evergreen Creek Catchment
- Flint Creek Catchment
- Golf Creek Catchment
- Greystone Creek Catchment
- Guadalupe Creek Catchment
- Hale Creek Catchment
- Lower Berryessa Creek Catchment
- Lower Silver Creek Catchment
- Matadero Creek Catchment
- Permanente Creek Catchment
- Regnart Creek Catchment
- Ross Creek Catchment
- San Tomas Creek Catchment
- South Babb Creek Catchment
- Stevens Creek Catchment
- Thompson Creek Catchment
- Yerba Buena Creek Catchment

N

0 Feet 30,000

Figure 7-1
Watershed Areas where Directly Connected Impervious Area will cause Additional Stream Segments to Destabilize

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P:\GIS\santaclaraWD\projects\May04HMP\Figure7-1

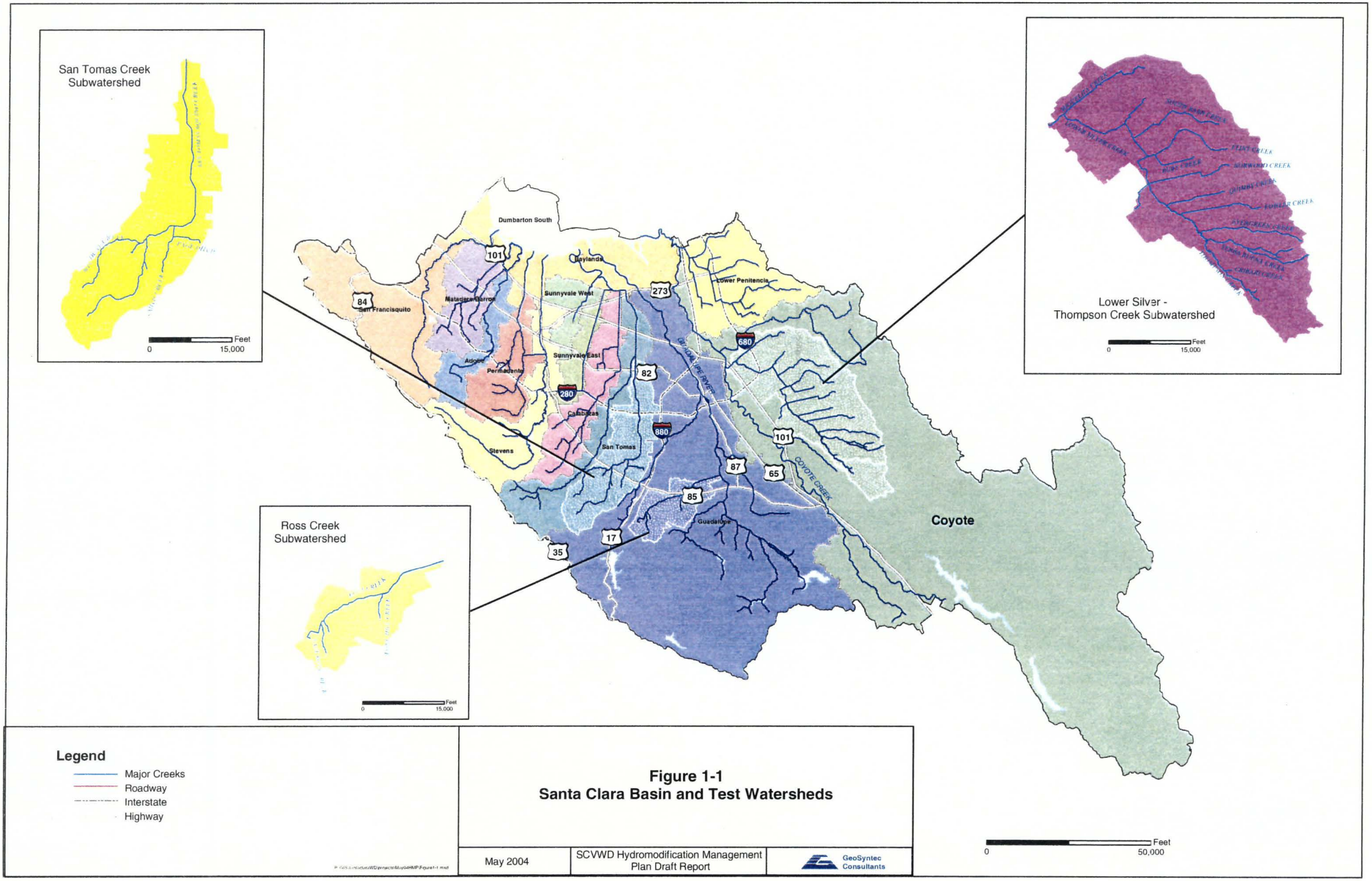


Figure 1-1
Santa Clara Basin and Test Watersheds

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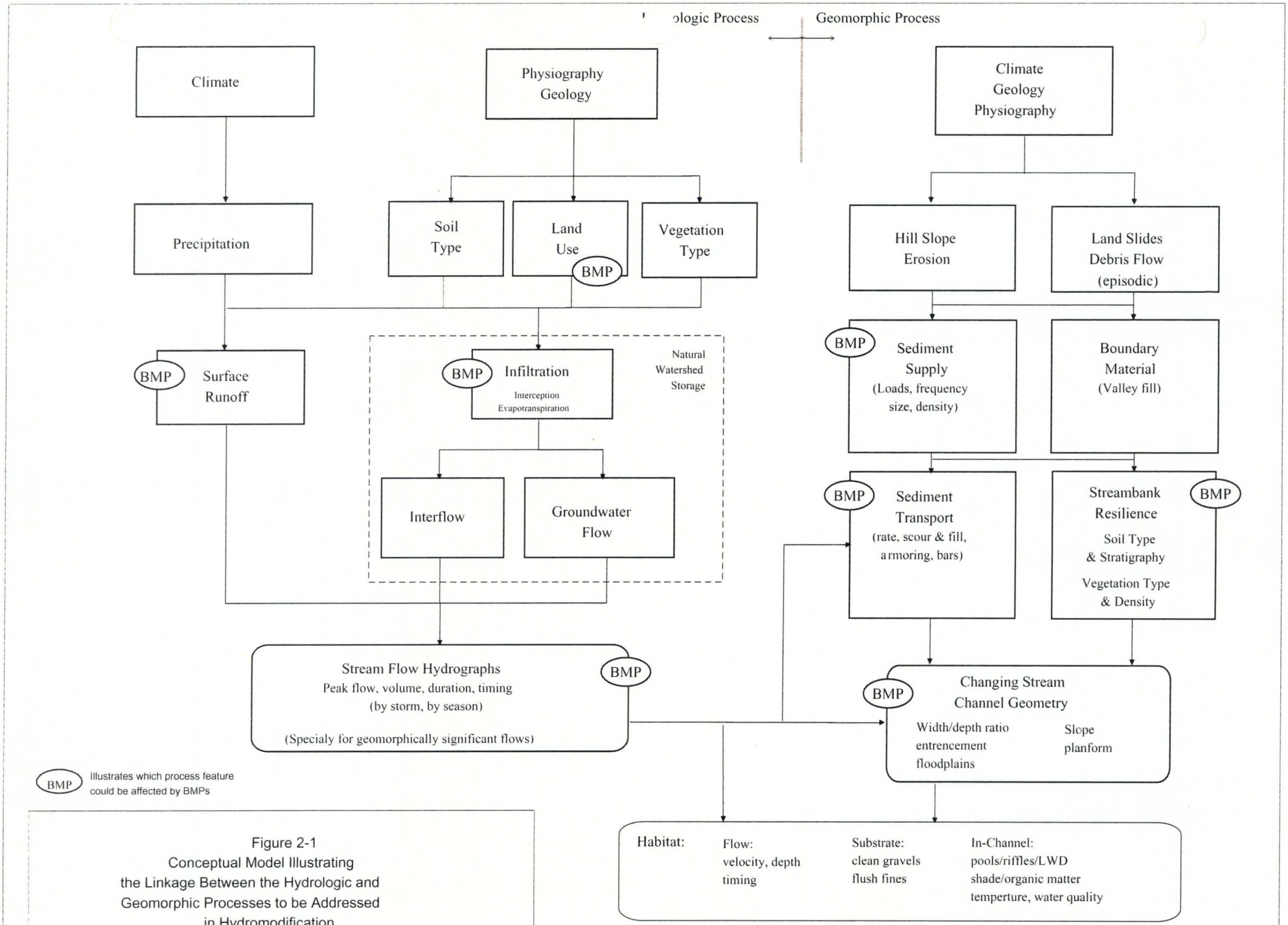
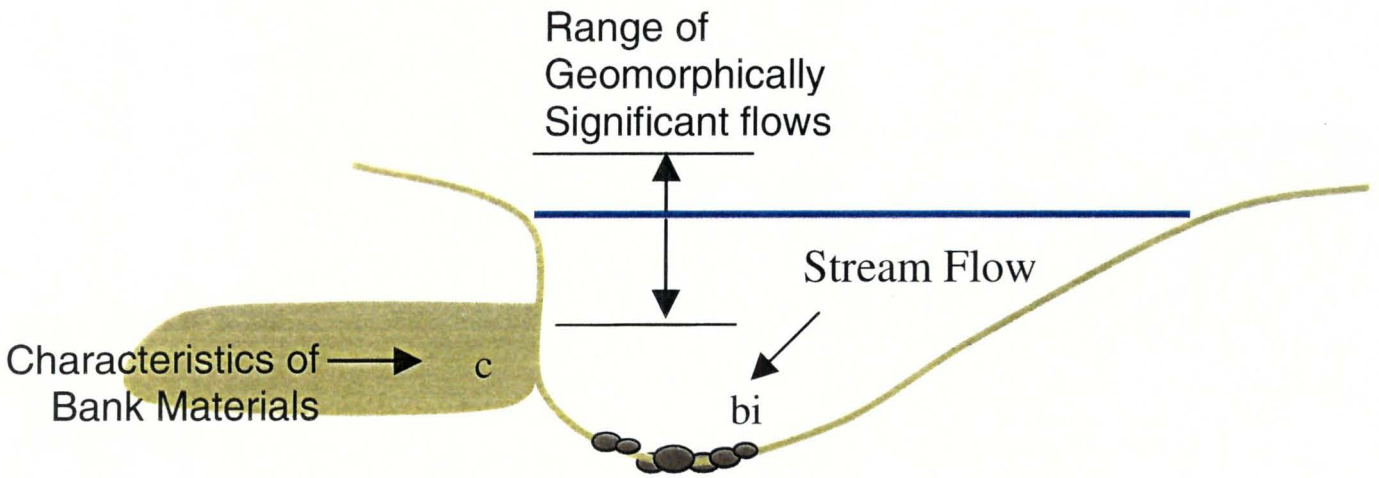


Figure 2-1
Conceptual Model Illustrating
the Linkage Between the Hydrologic and
Geomorphic Processes to be Addressed
in Hydromodification



$$W \sum_{i\ 1}^{n} {}_{bi} \quad {}_{c} \quad V^{1.5} \quad t$$

Figure 2-2
Schematic Illustrating Effective Work Index

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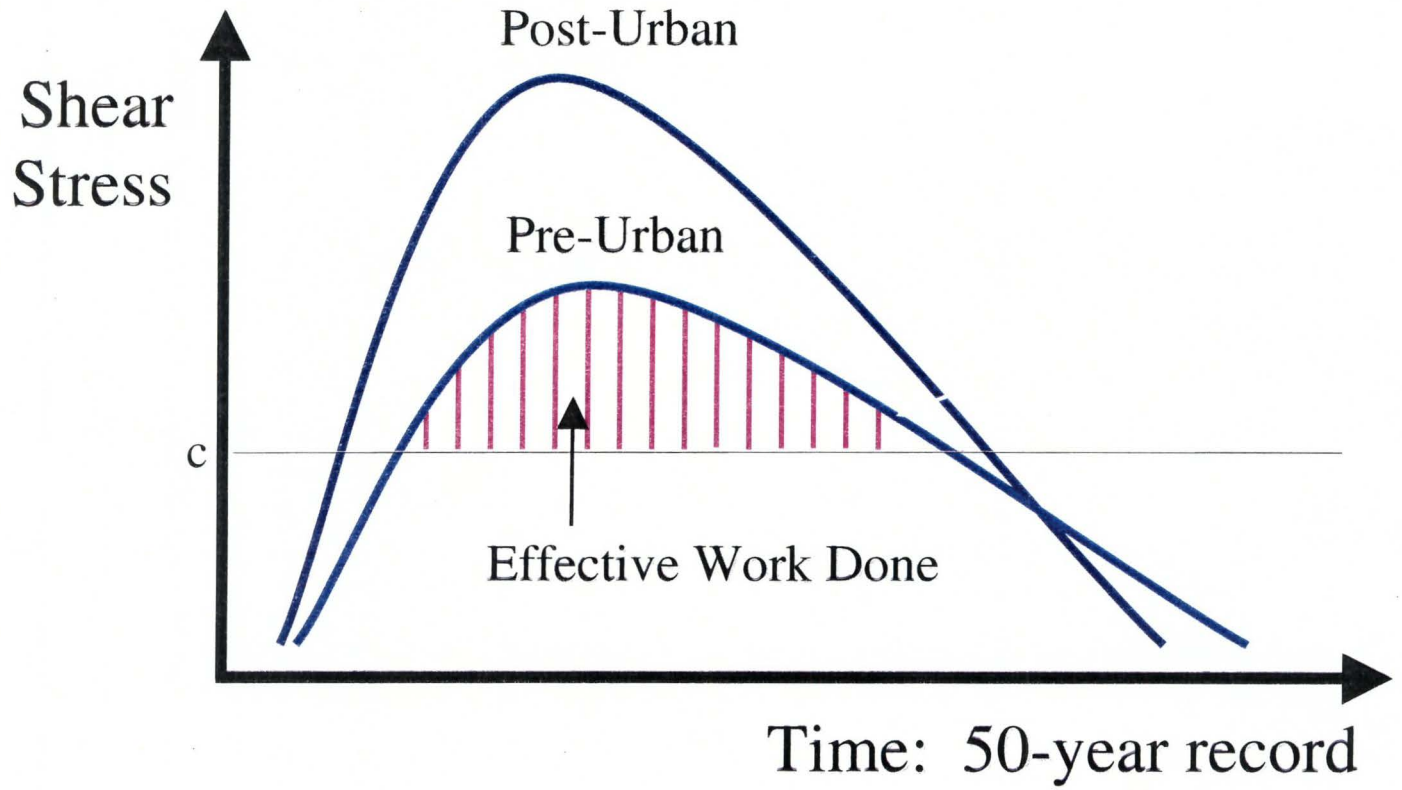


Figure 2-3
Definition of Ep Ratio

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Figure 3-1
Cross-section survey at YB1-4. A tape was stretched between two pins to measure the channel dimensions,bed material, bank material, vegetation, roughness and other characteristics were described.



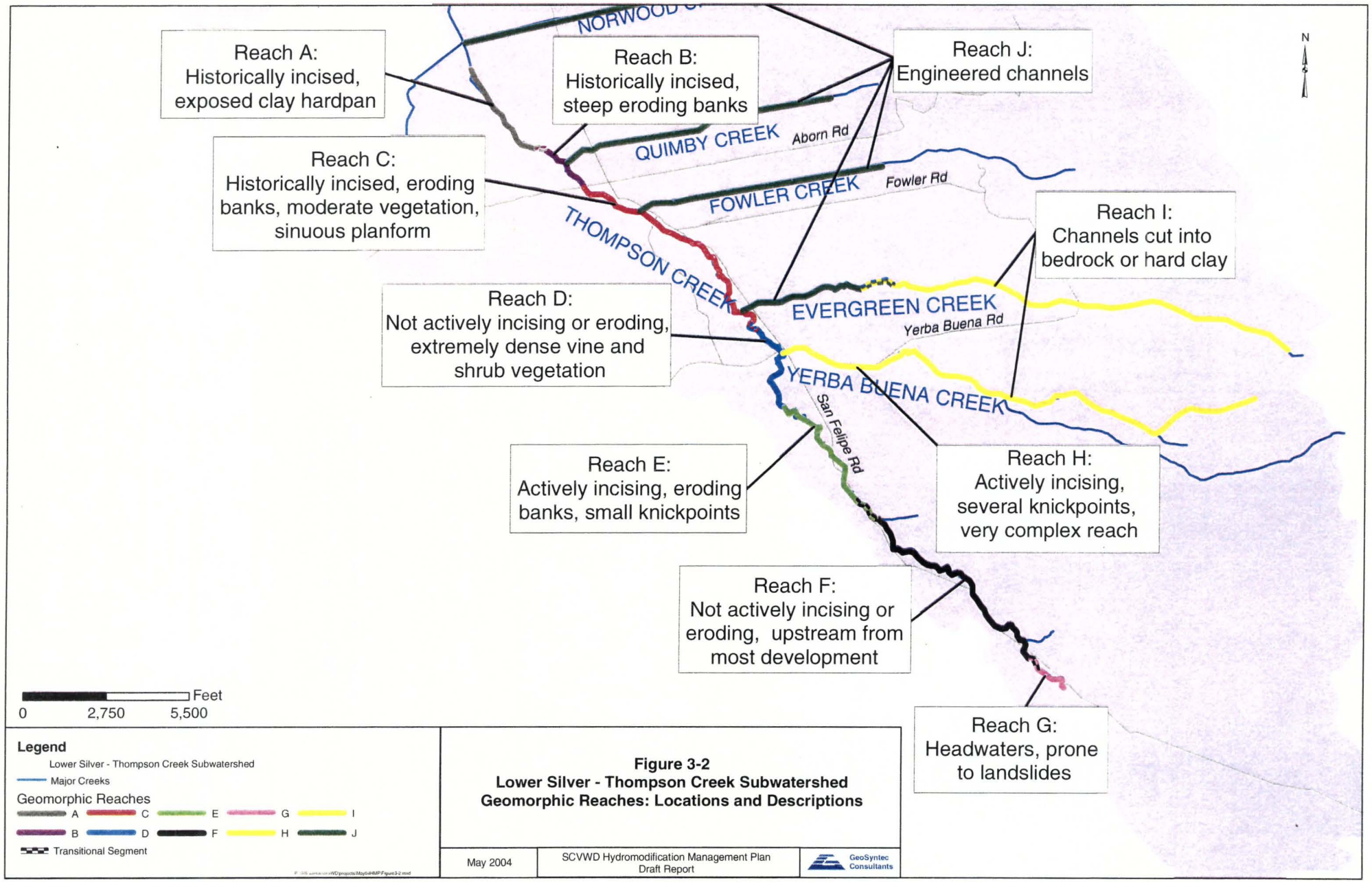


Figure 3-2
Lower Silver - Thompson Creek Subwatershed
Geomorphic Reaches: Locations and Descriptions

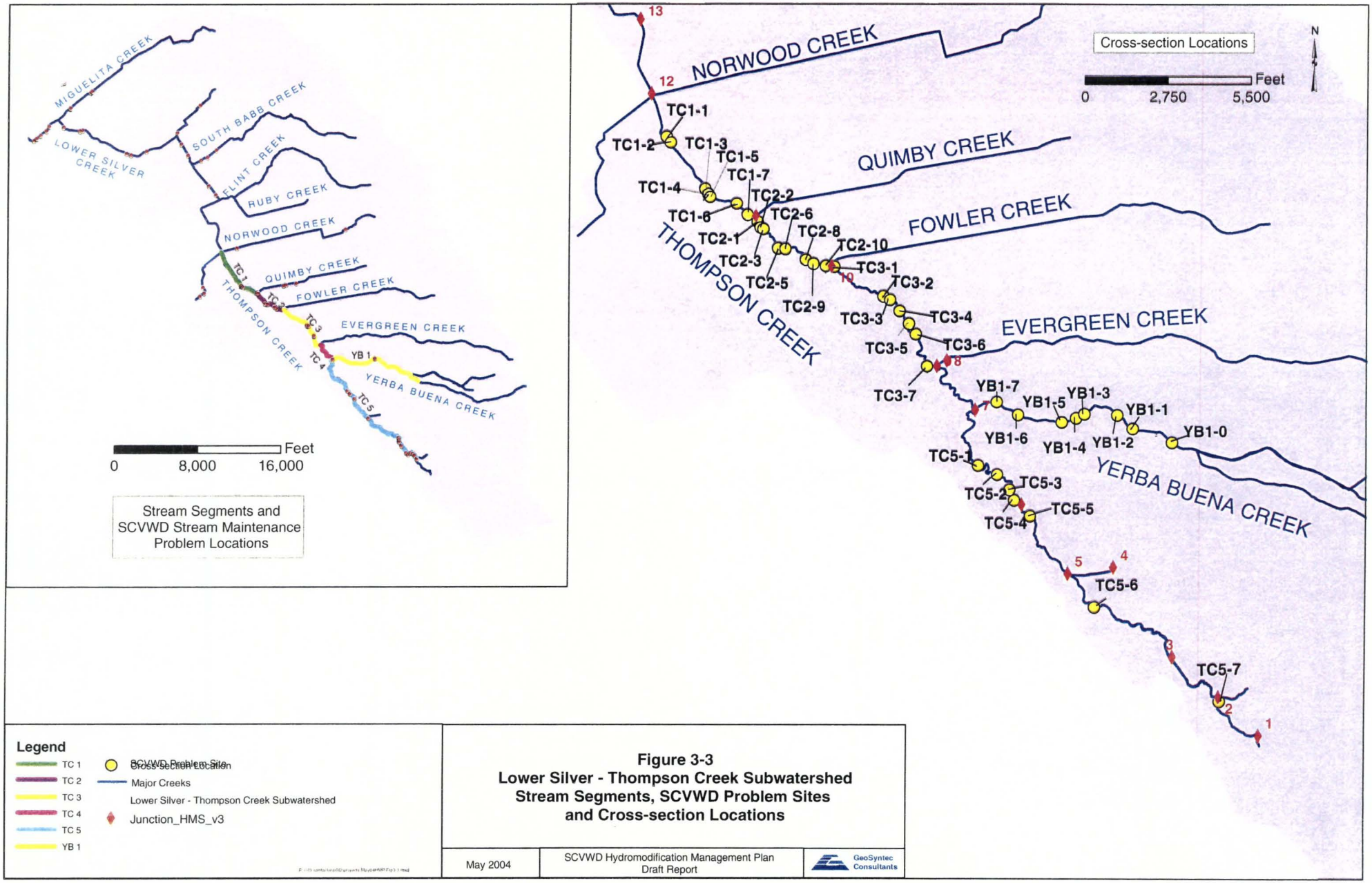


Figure 3-3
Lower Silver - Thompson Creek Subwatershed
Stream Segments, SCVWD Problem Sites
and Cross-section Locations



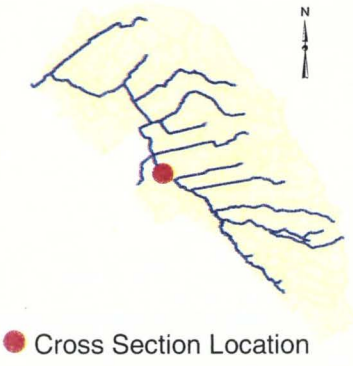
Figure 3-4
Photo showing a large grade control structure in Segment TC1. Note the exposed clay-silt hardpan just down stream from the structure.



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Figure 3-5
Photo of the bank material in Segment TC1, near cross-section TC1-3. Note that the lower unit in the bank erodes more easily than the upper layer.



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Figure 3-6
The upper portion of segment TC2 shows less incision than below Aborn Road. The banks are stabilized by tree roots and blackberries, though some bank erosion is still occurring (Photo taken at cross-section TC2-9).



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Figure 3-7
Segment TC3 is similar to Segment TC2, but with slightly more incision.



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Figure 3-8
Photo at cross-section TC5-5. Note the dense vegetation, relatively straight channel, and exposed root mat near water level.



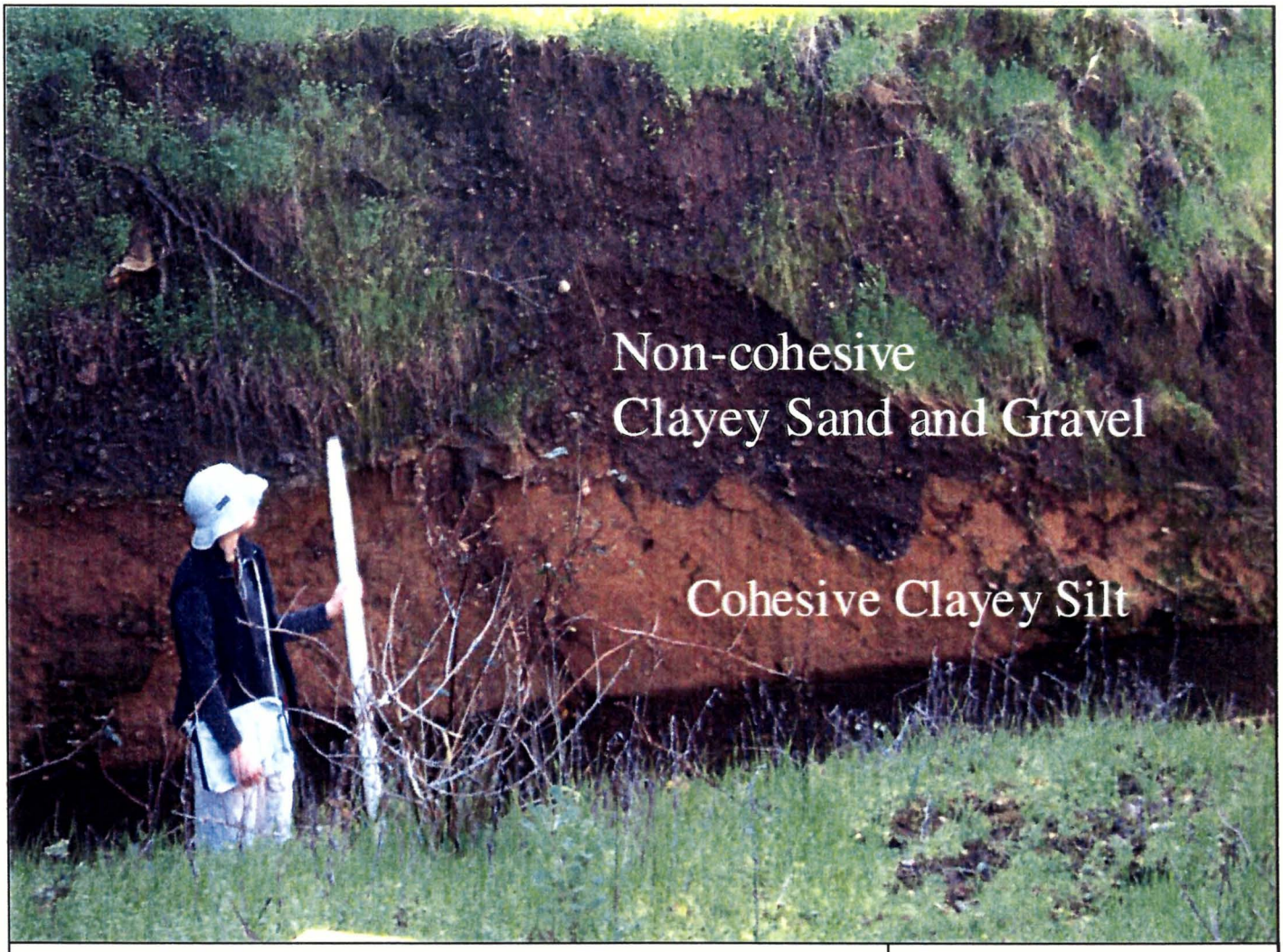


Figure 3-9
Photo showing contact between cohesive clayey silt (bottom of bank) and non-cohesive clayey sand and gravel (upper portion of bank). Note that the upper portion of the bank is partially slumped.



Cross Section Location



Figure 3-10
Photo of deeply incised portion of Yerba Buena Creek at cross-section YB1-6. The creek has incised well below the majority of the roots in the bank.



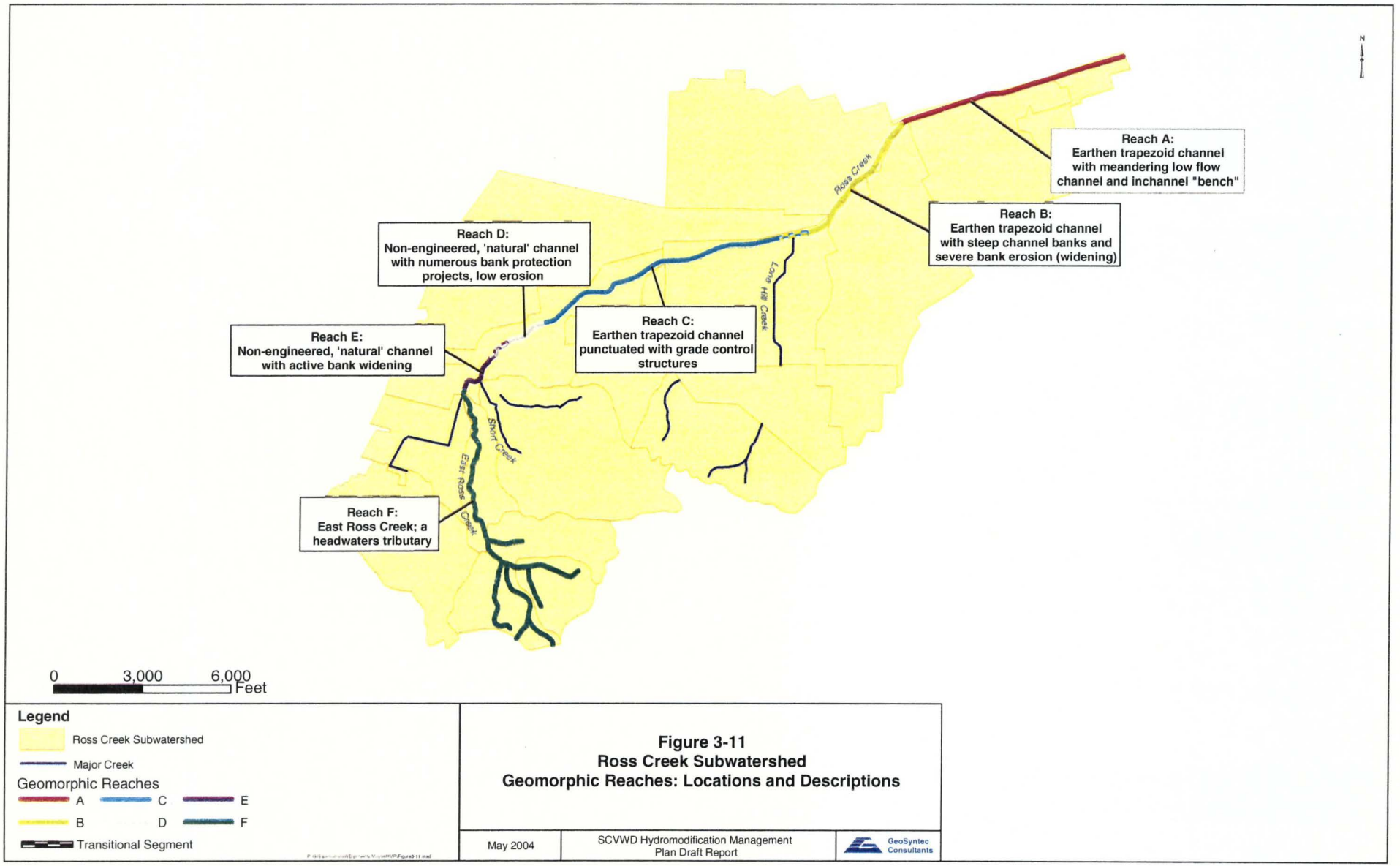


Figure 3-11
Ross Creek Subwatershed
Geomorphic Reaches: Locations and Descriptions



Legend

- Cross Section Location
- Problem site
- Outfall
- Creek
- Main
- Lateral
- Catchment Boundary
- Watershed Boundary

N

0 2,500 5,000 Feet

Figure 3-12
Ross Creek Subwatershed
SCVWD Problem Sites and
Cross Section Locations

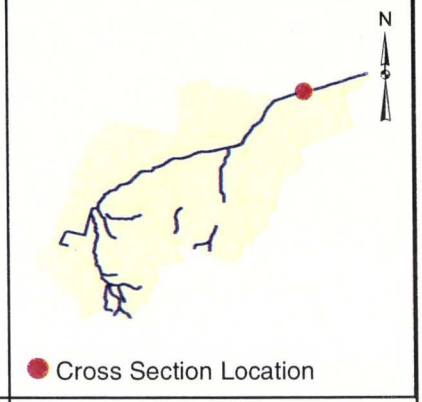
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Figure R3-2
Linear bench that is prevalent throughout Geomorphic Reaches A and C, shown here at cross-section RC-2, Ross Creek.



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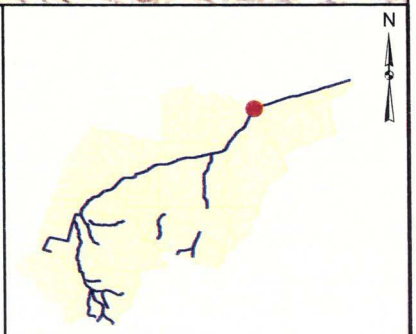
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Figure R3-3
Actively eroding channel banks characteristic of Geomorphic Reach B, looking across cross-section RC-3, Ross Creek.



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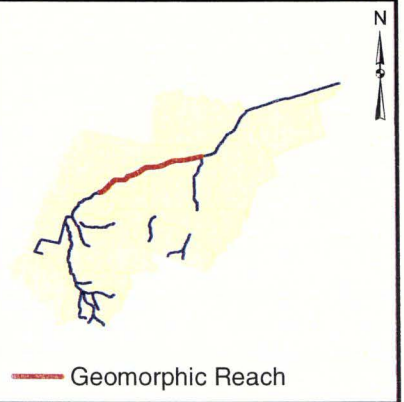
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Figure R3-4
One of several grade control structures located in Geomorphic Reach C, this one is located upstream from Camden Road and downstream from cross-section RC-4, Ross Creek.



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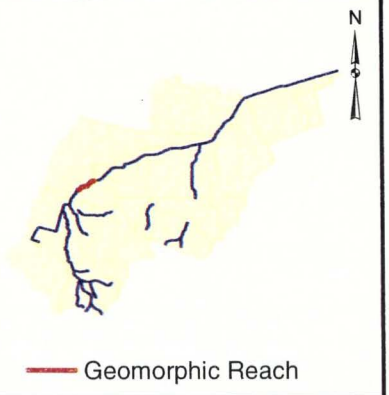
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Figure R3-5
Large trees are present along the banks of Ross Creek Reach D.



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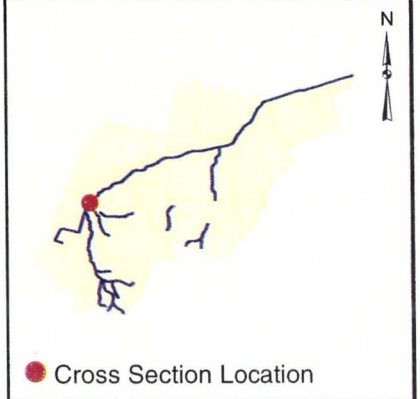
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Figure R3-6
Channel bank widening leaving tree roots exposed at cross-section RC-8 in Geomorphic Reach E, Ross Creek.



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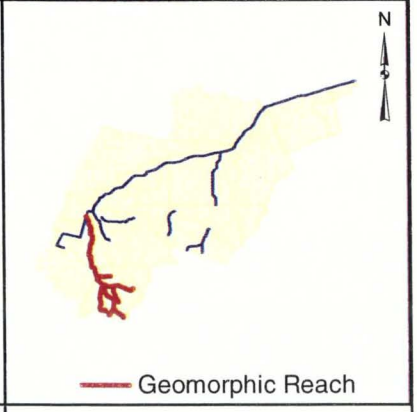
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Figure R3-7
Upper portion of East Ross Creek,
Geomorphic Reach F.



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Figure 3-13
San Tomas Creek Subwatershed
SCVWD Problem Sites and
Cross Section Locations

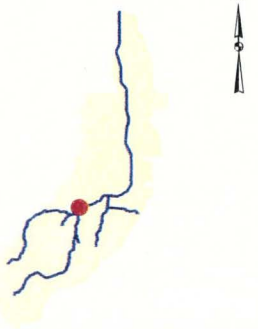
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Figure S3-2
Cross-section ST-3 was ranked as a "low-erosion" site, San Thomas Creek.



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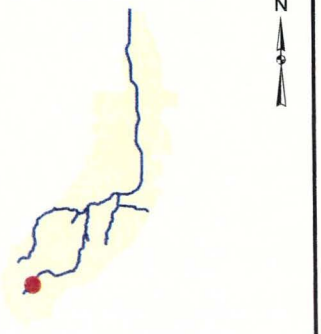
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Figure S3-3
Boulders in channel bed at cross-section ST-12, San Thomas Creek.



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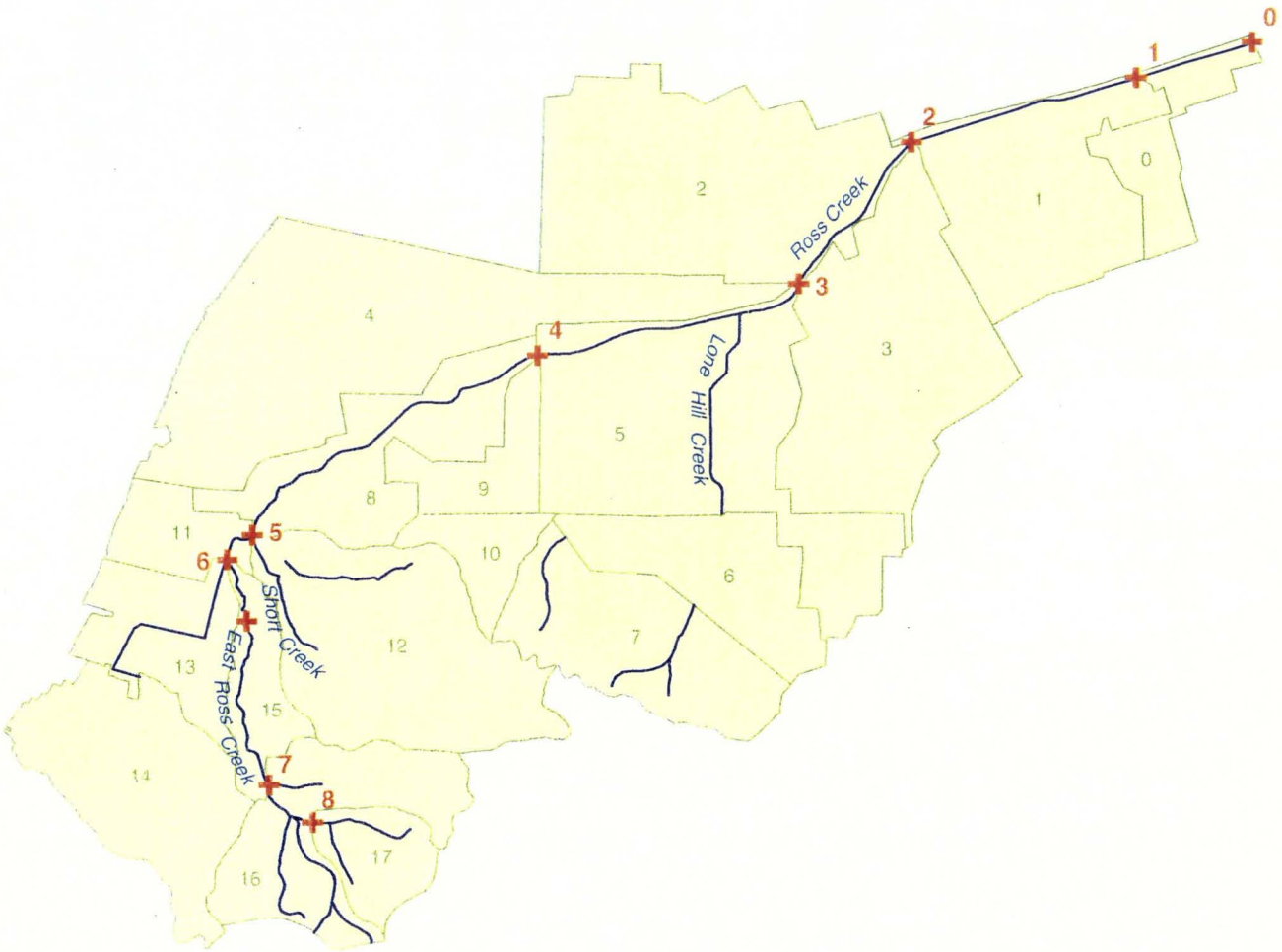
Legend

- Junction Locations and Names
- Drainage Areas
- Main Roads

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Figure 4-1
Drainage Area Delineations for the Lower Silver - Thompson Creek Subwatershed

(Note: Creeks have been shown above ground but actually run underground in some of the urban areas)



Feet
0 4,000

Legend

- Junction Location
- Drainage Area
- Major Creek

Figure 4-2
Drainage Area Delineations for the Ross Creek Subwatershed
(Note: Creeks have been shown above ground but actually run underground in some of the urban areas)

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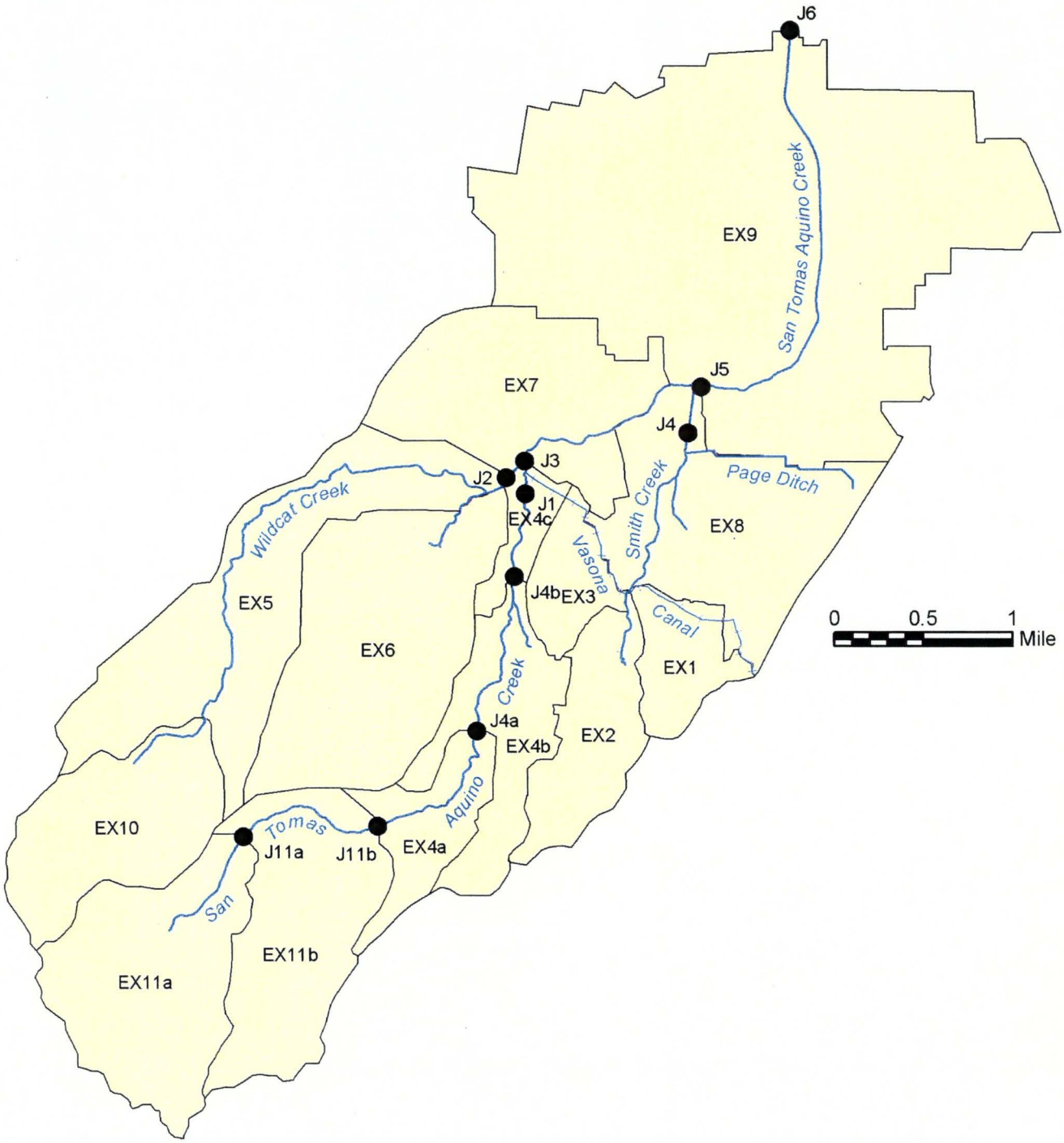
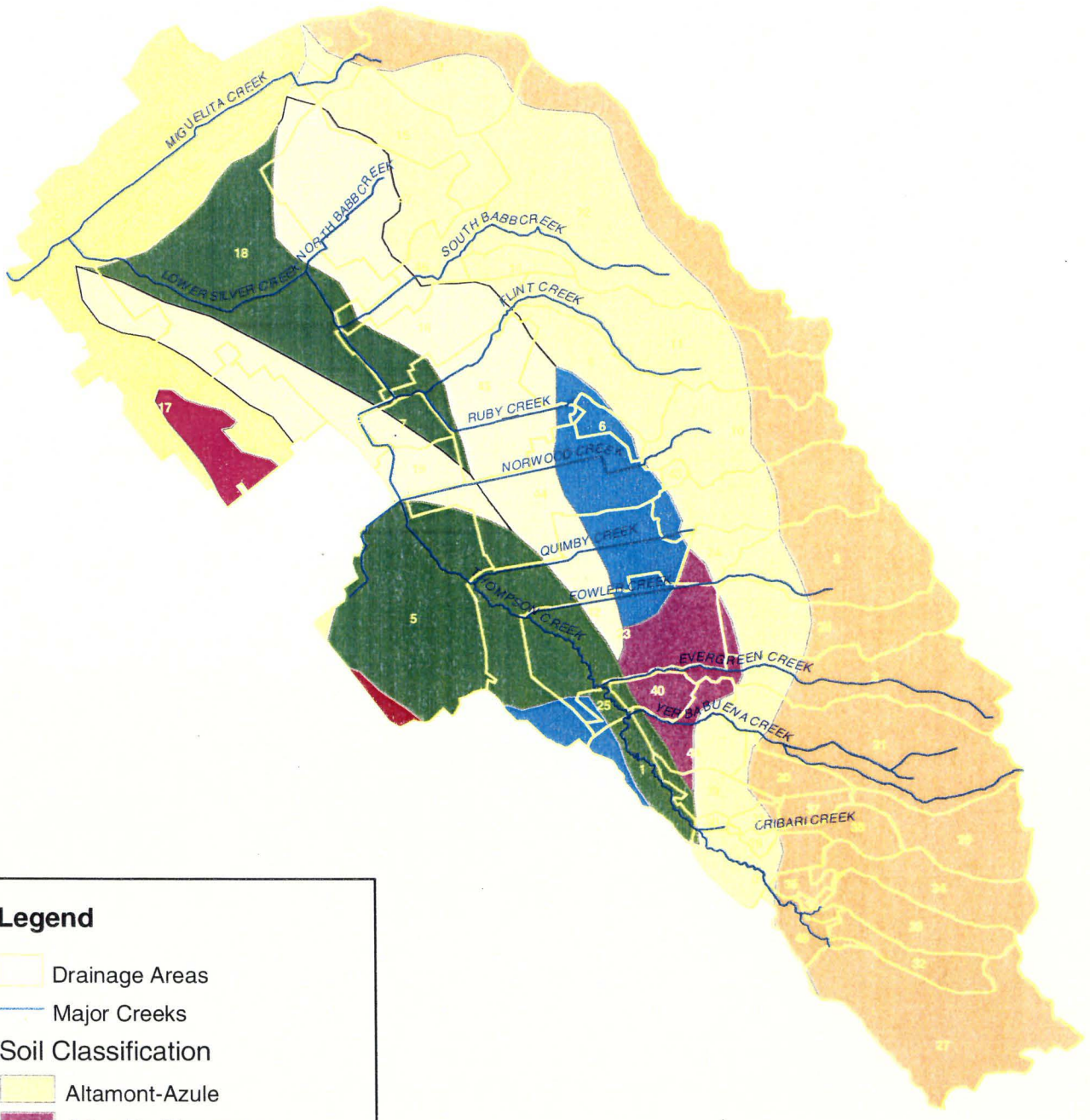


Figure 4-3
Drainage Area Delineations for the San Tomas Subwatershed

Legend

- Connection
- Stream
- Canal
- Basin

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Legend

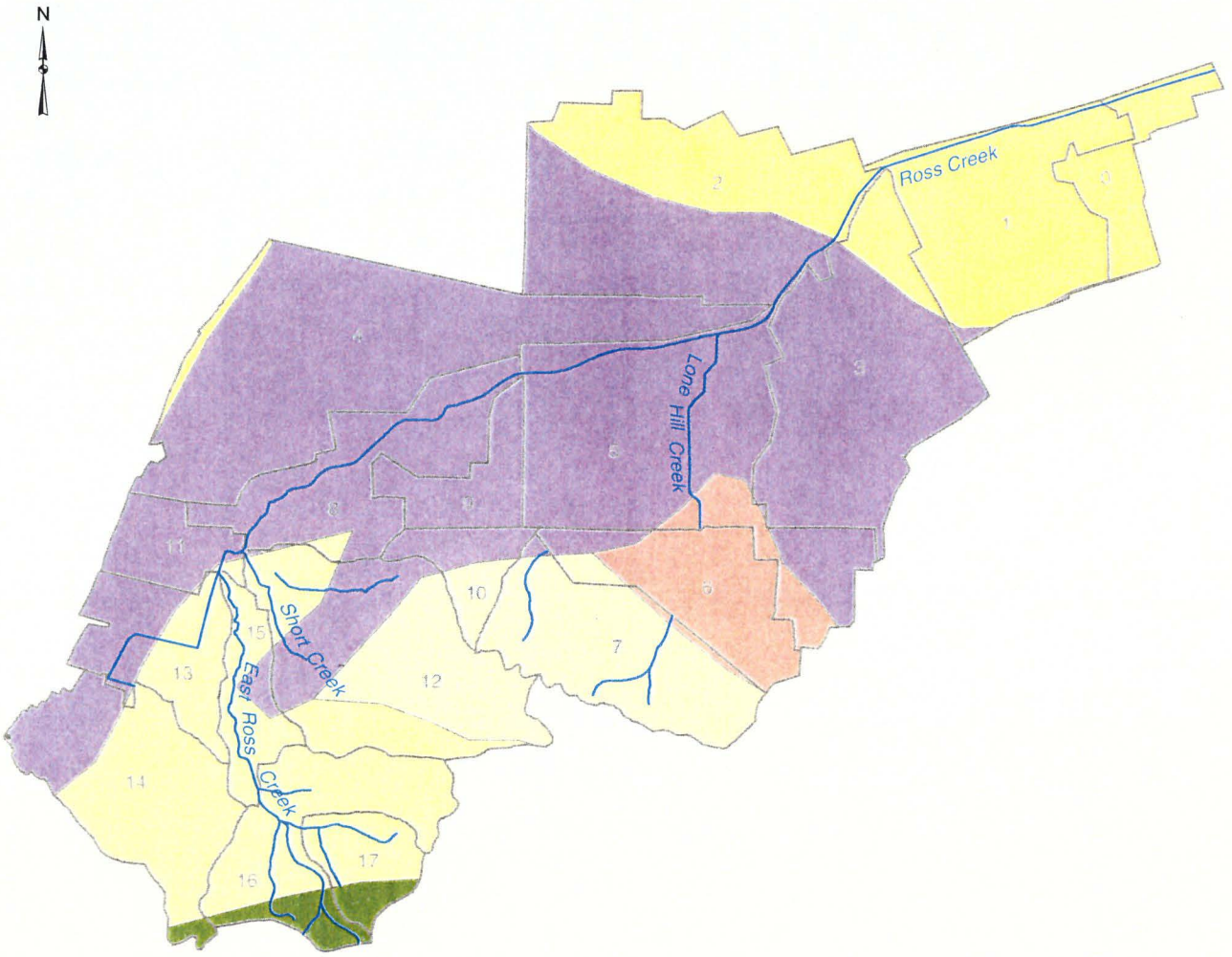
- Drainage Areas
- Major Creeks

Soil Classification

- Altamont-Azule
- Arbuckle-Pleasanton
- Clear Lake-Campbell
- Cropley-Rincon
- Hillgate-Positas
- Los Gatos-Gaviota-Vallecitos
- Orestimba-Willows
- Sunnyvale-Castro-Clear Lake
- Yolo
- Zamora-Pleasanton

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Figure 4-4
Soil Classifications for the
Lower Silver - Thompson Creek Subwatershed



Legend

Drainage Area

Major Creek

Soil Classification

- Altamont-Azule
- Arbuckle-Pleasanton
- Clear Lake-Campbell
- Los Gatos-Gaviota-Vallecitos
- Maymen-Los Gatos-Gaviota
- Yolo

Feet
0 2,000 4,000

Figure 4-5
Soil Classifications for the Ross Creek Subwatershed

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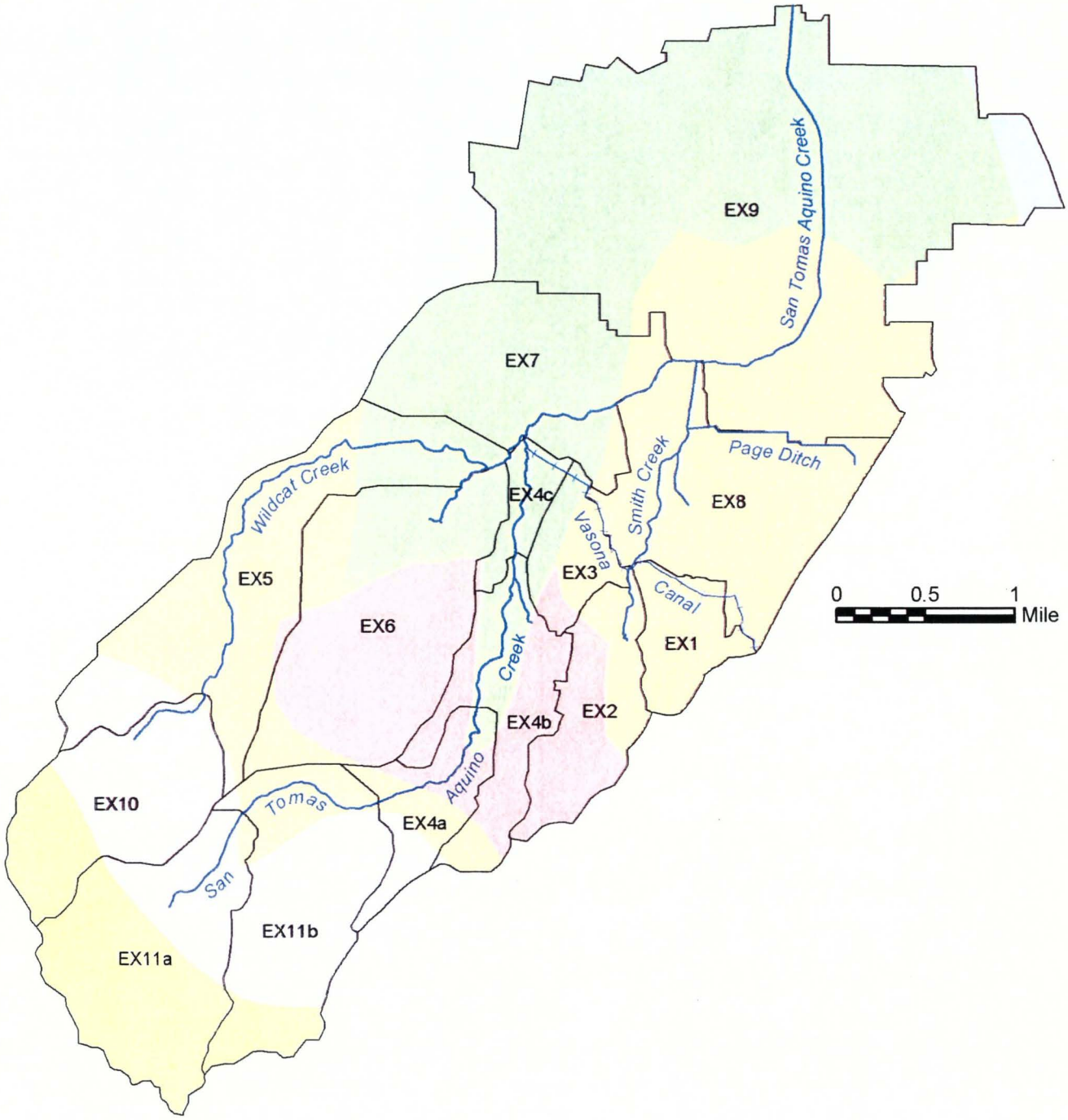
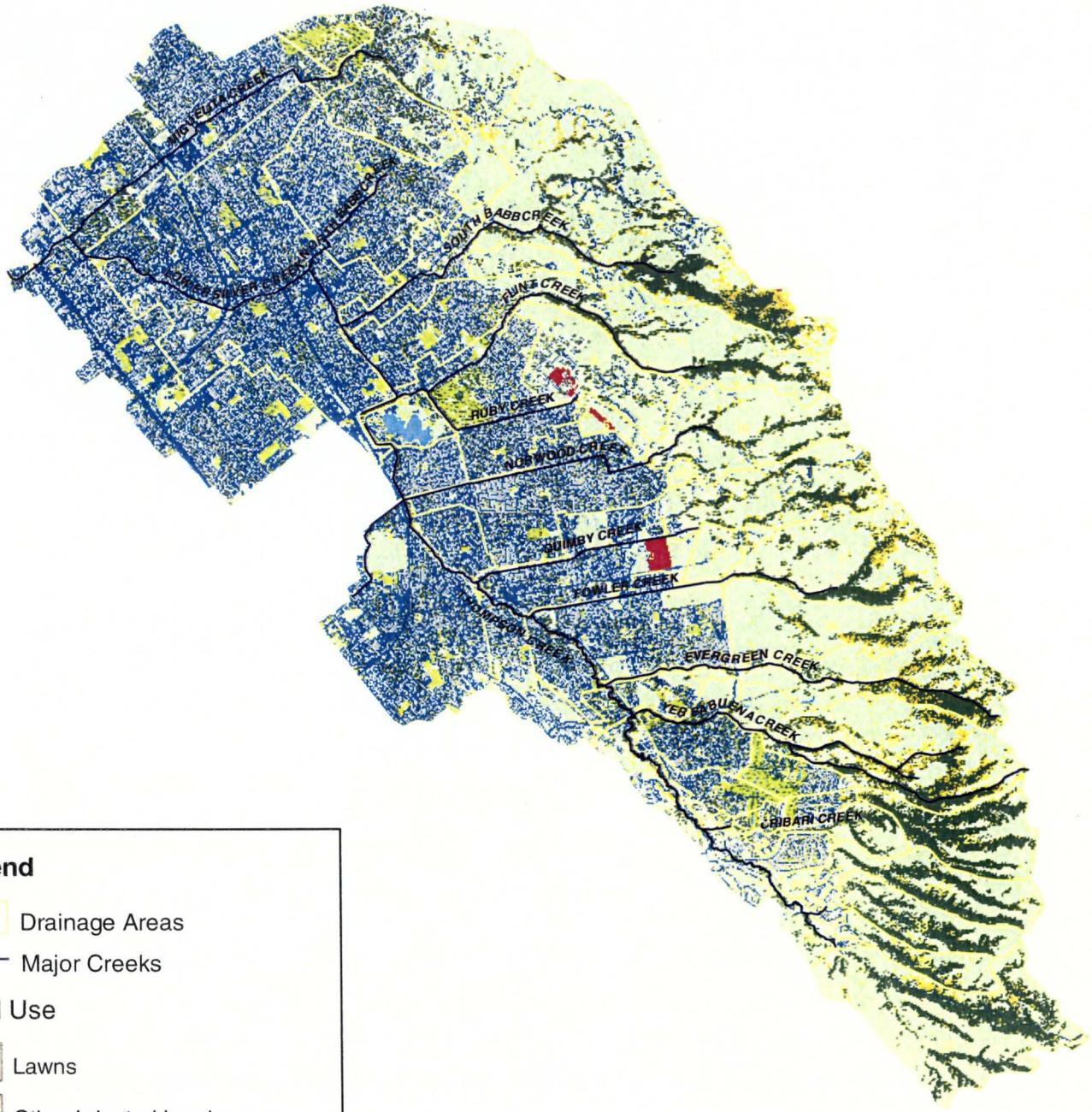


Figure 4-6
Soil Classification for the
San Tomas Subwatershed

Legend

- Arbuckle-Pleasanton
- Hillgate-Positas
- Hillgate-Soper
- Maymen-Los Gatos-Gaviota
- Yolo
- Zamora-Pleasanton

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Legend

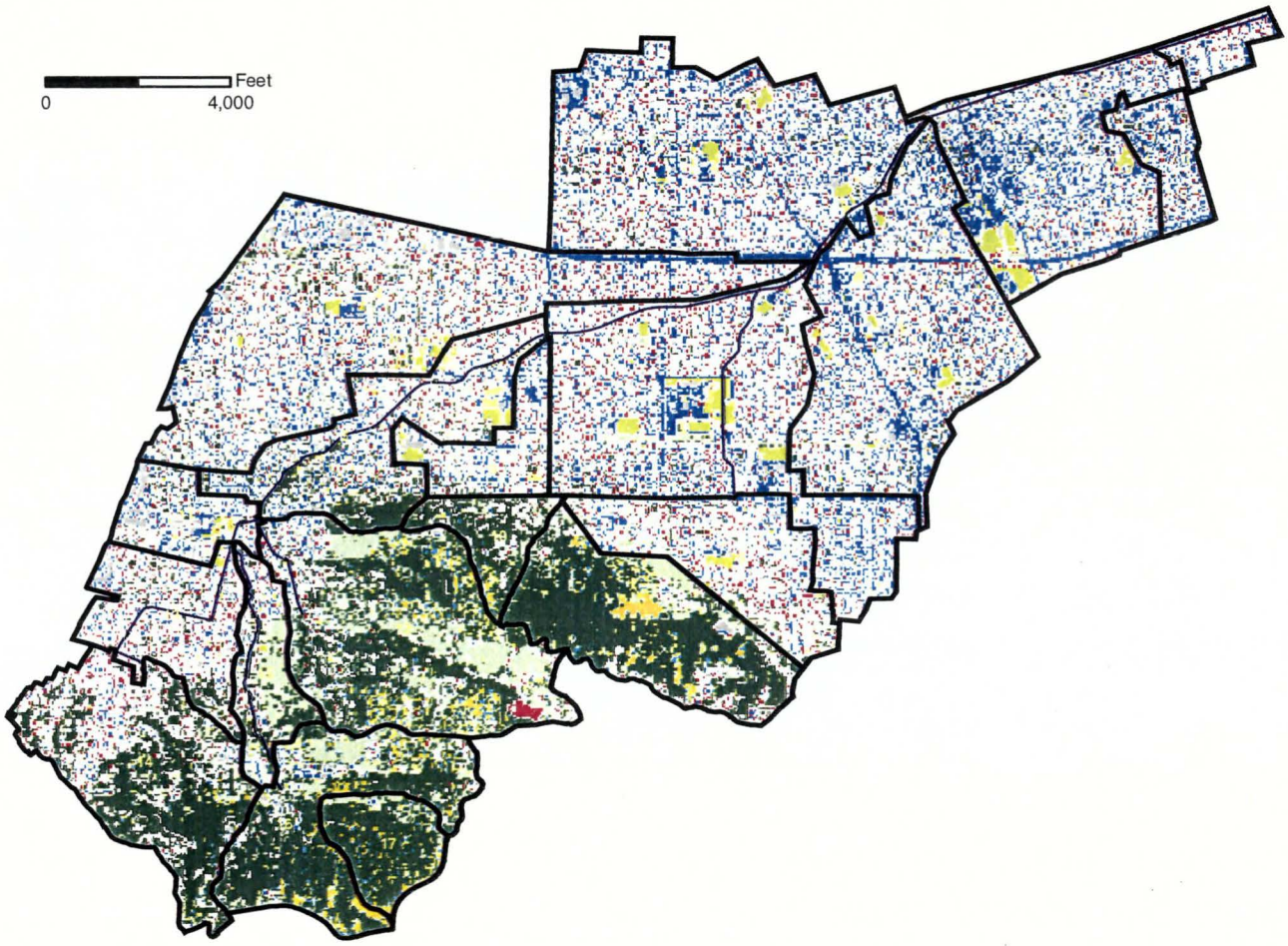
- Drainage Areas
- Major Creeks

Land Use

- Lawns
- Other Irrigated Landscapes
- Structures
- Paved Surfaces
- Agricultural Land Cover
- Grassland
- Trees
- Bare Ground
- Water Features
- Shrubs/ herbaceous

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Figure 4-7
Land Use for the
Lower Silver - Thompson Creek Subwatershed



Legend

N

Drainage Area

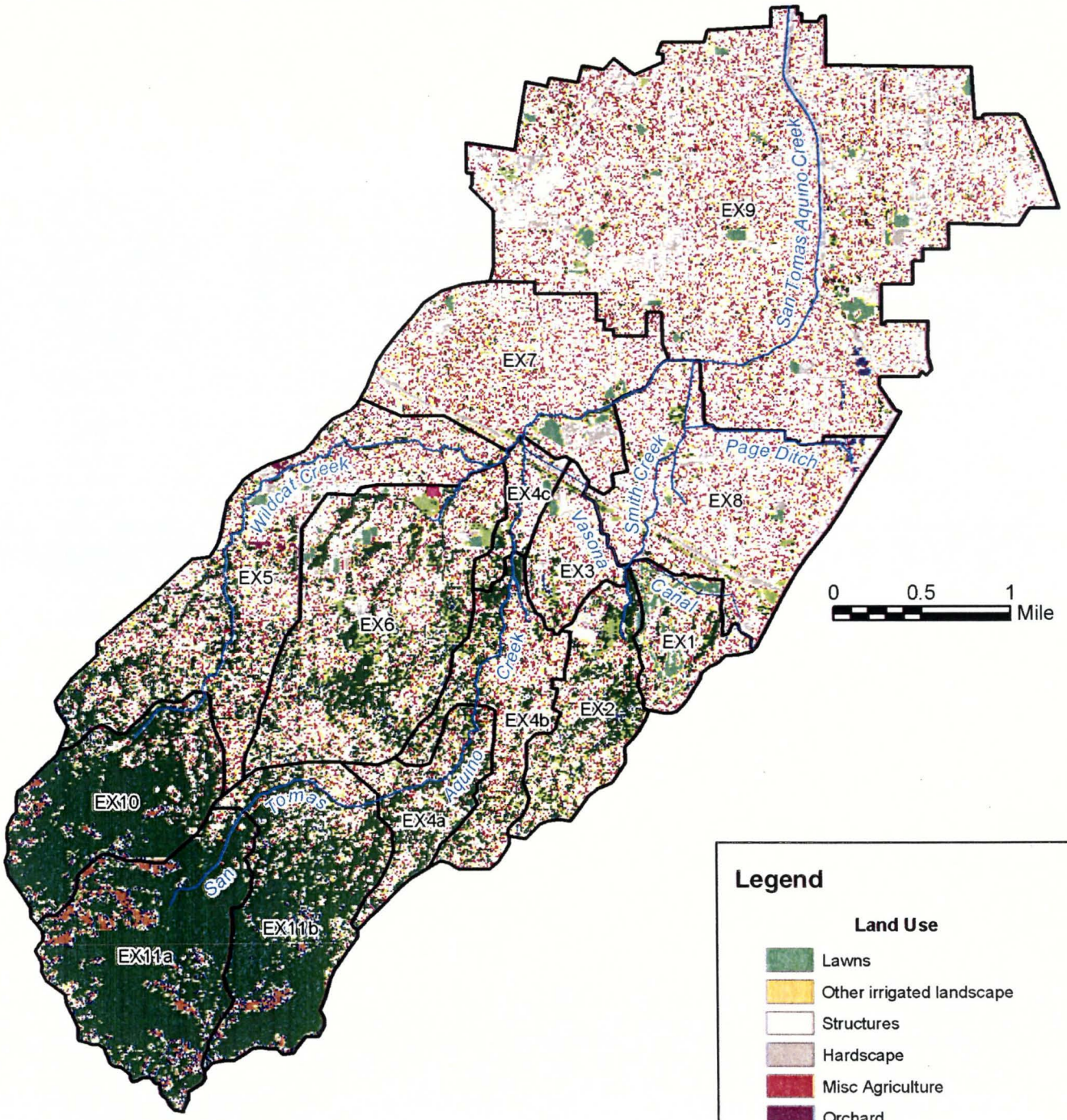
Major Creek

Land Use

- Lawns
- Other Irrigated Landscapes
- Structures
- Paved Surfaces
- Agricultural Land Cover
- Grassland
- Trees
- Bare Ground
- Water Features
- Shrubs/ herbaceous

Figure 4-8
Land Use for the Ross Creek Subwatershed

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Legend

Land Use

- Lawns
- Other irrigated landscape
- Structures
- Hardscape
- Misc Agriculture
- Orchard
- Vegetables, flowers
- Row crops
- Vineyards
- Irrigated pasture
- Ornamental Eucalyptus
- Alfalfa
- Grassland (non-irrigated)
- Trees
- Bare/fallow
- Water
- Shrubs/herbacious/wetland
- Greenhouse

Figure 4-9
Land Use for the
San Tomas Subwatershed

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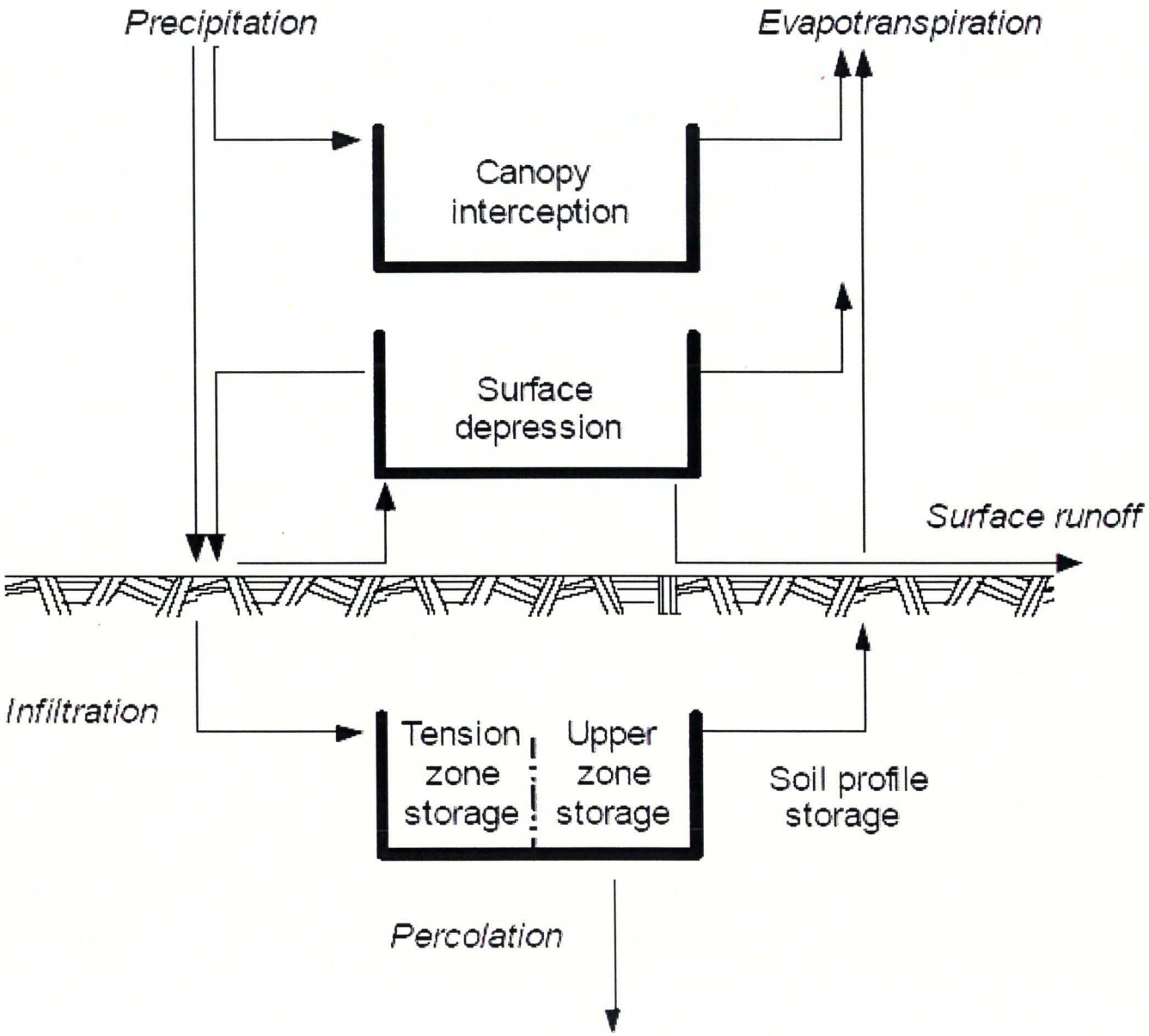


Figure 4-10
Schematic of HEC-HMS Soil Moisture Accounting Model

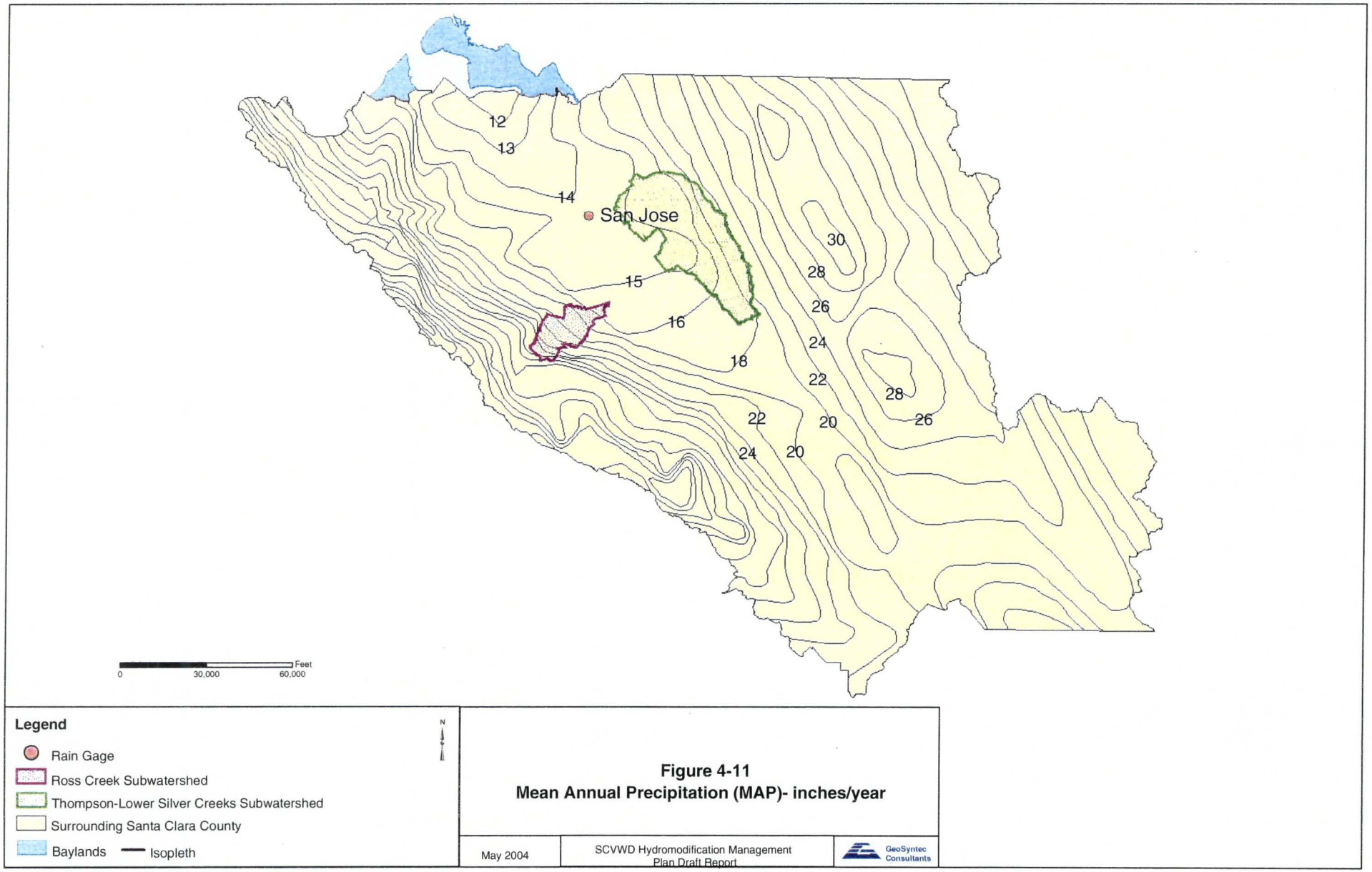


Figure 4-11
Mean Annual Precipitation (MAP)- inches/year

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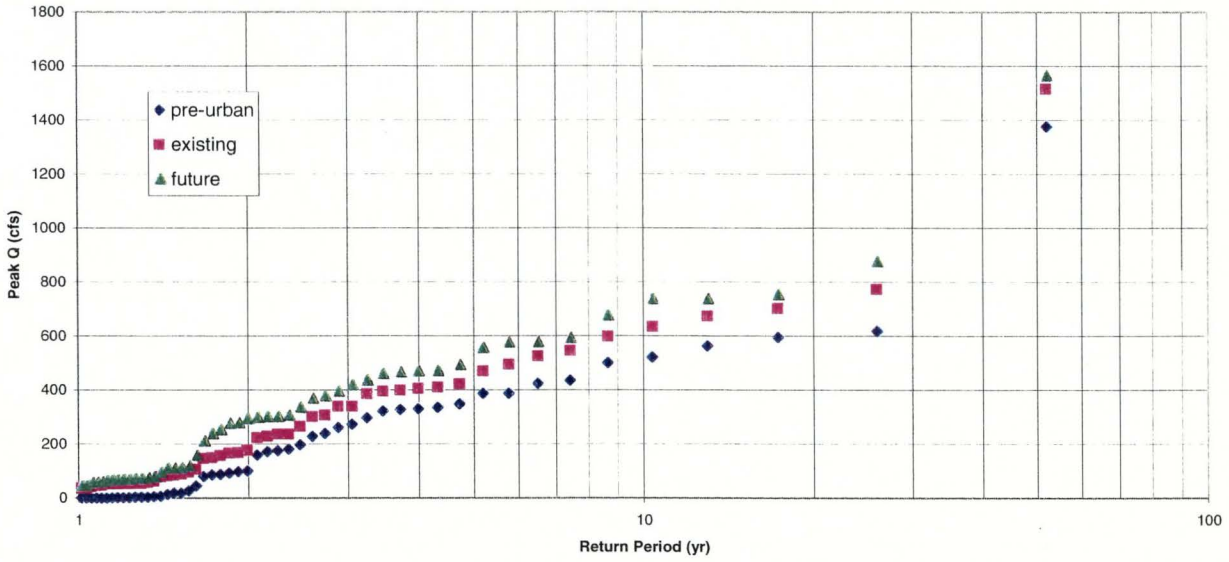
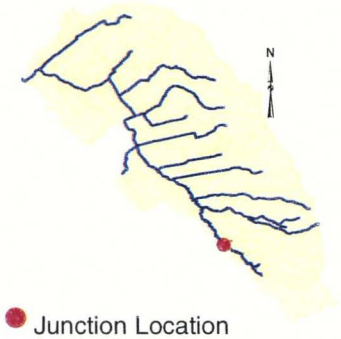


Figure 4-12
Flood Frequency for Junction J-5
Using Continuous Model Results



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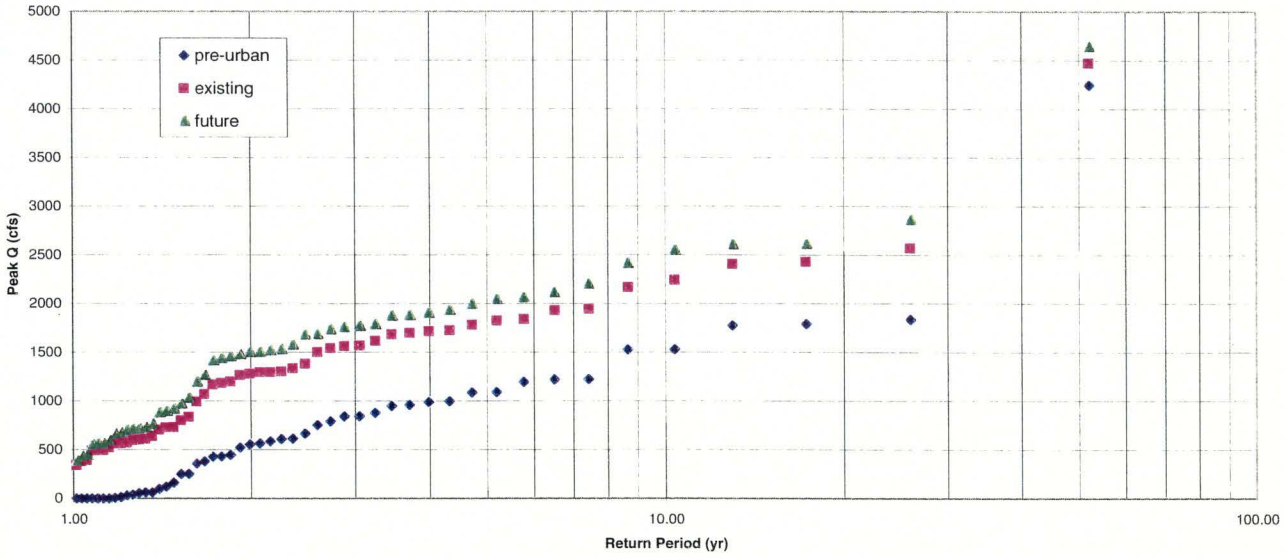
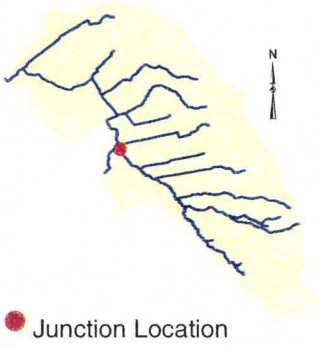


Figure 4-13
Flood Frequency for Junction J-12
Using Continuous Model Results



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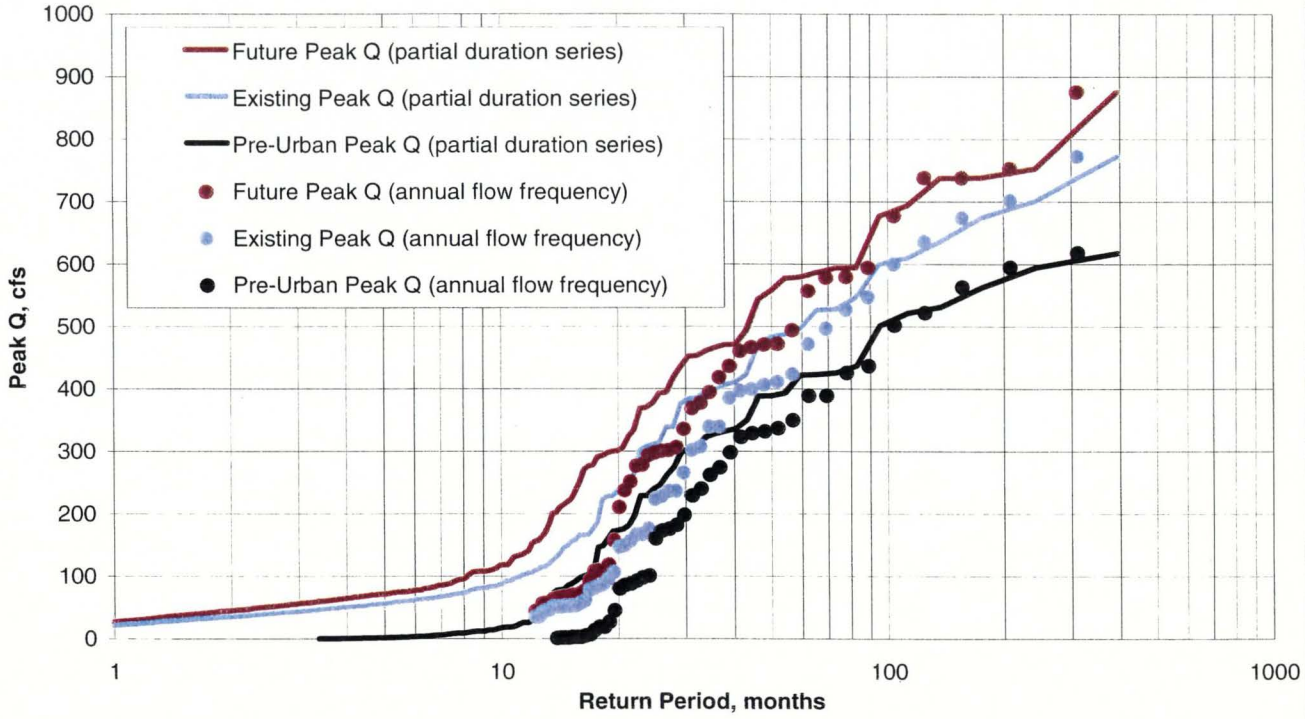
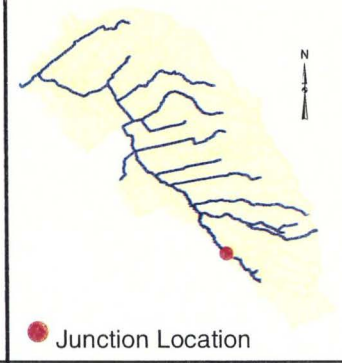


Figure 4-14
Peak Flow Frequency Analysis: Monthly and Annual Recurrence Intervals at Location J-5



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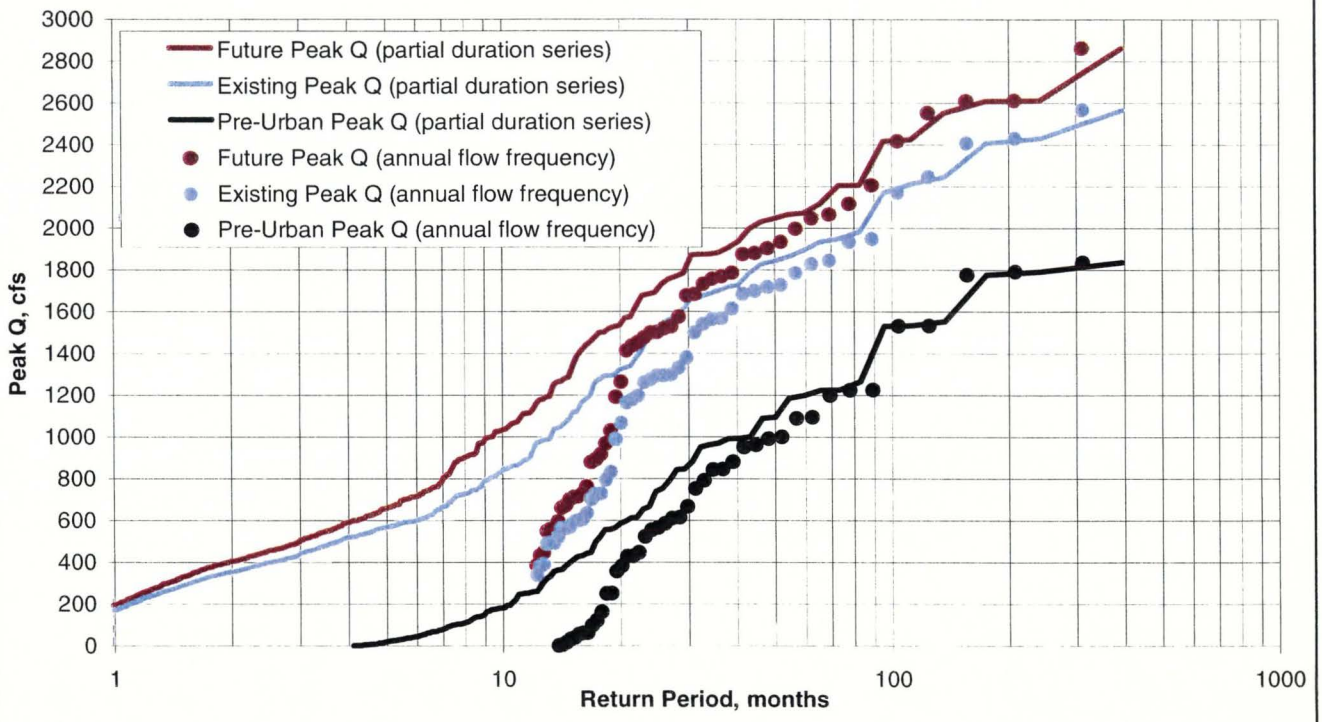
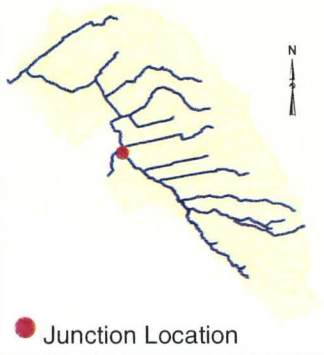


Figure 4-15
Peak Flow Frequency Analysis: Monthly and Annual Recurrence Intervals at Location J-12



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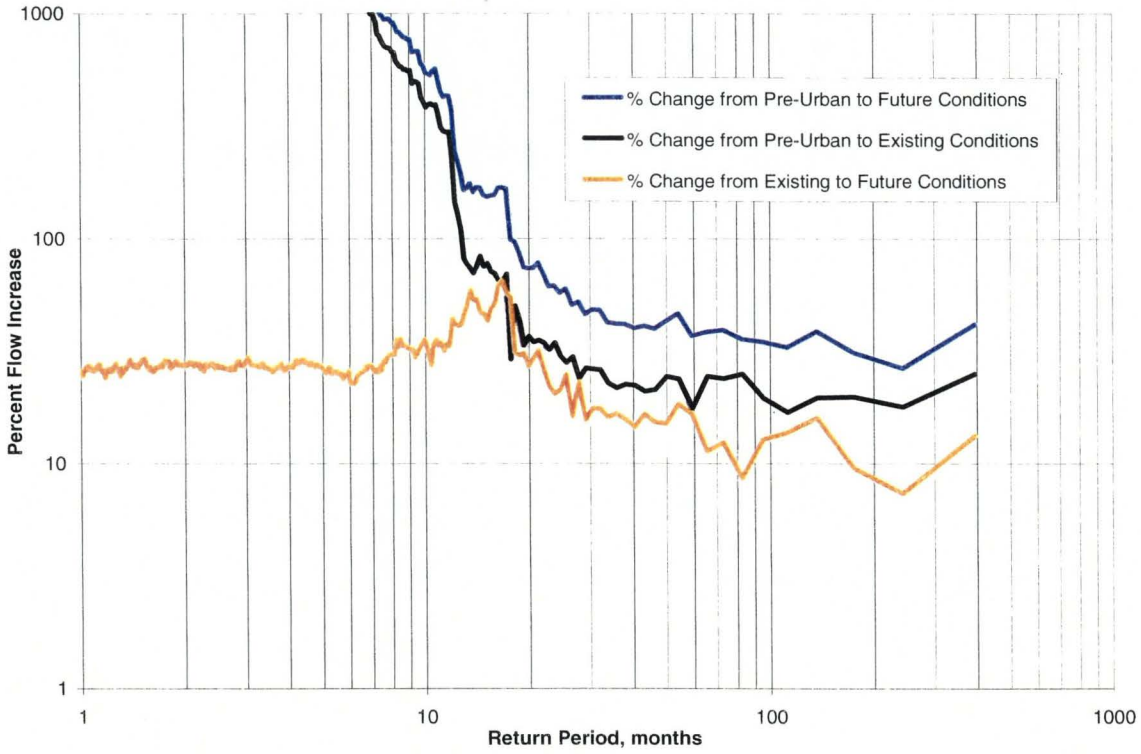
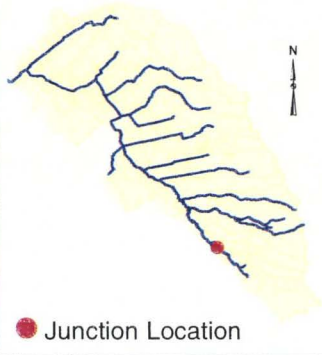


Figure 4-16
Percent Flow Increase at Location J-5



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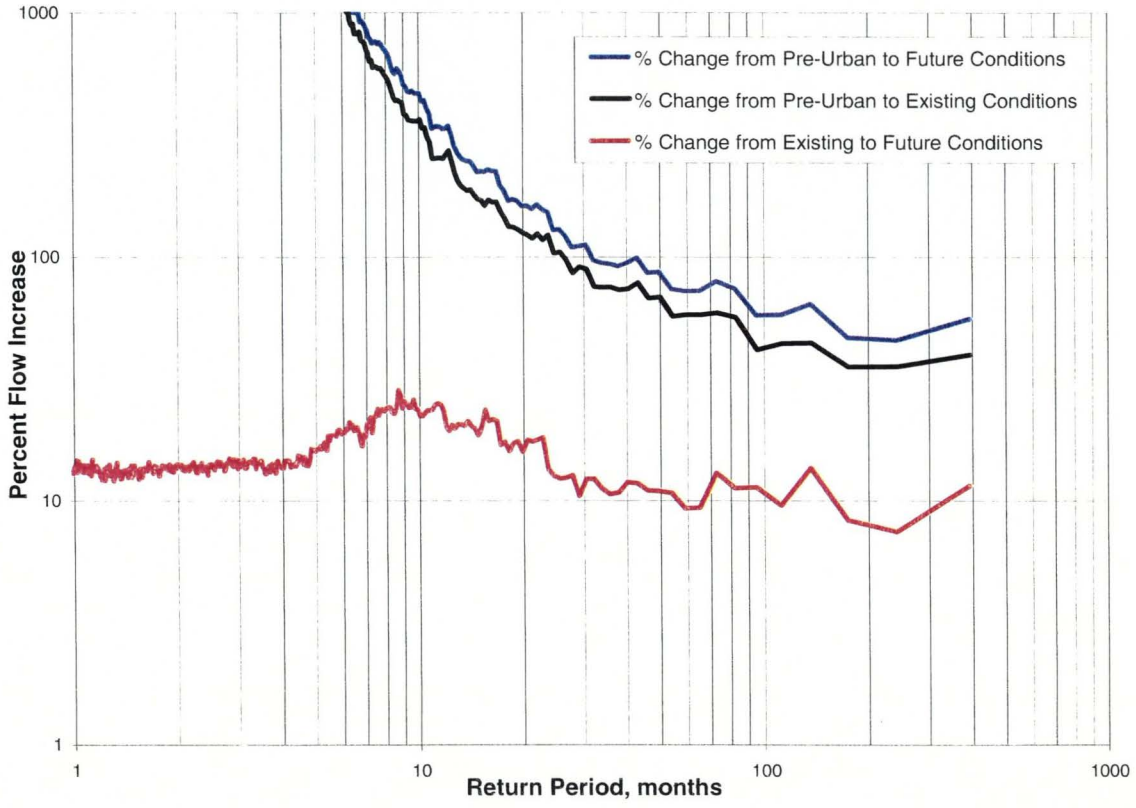
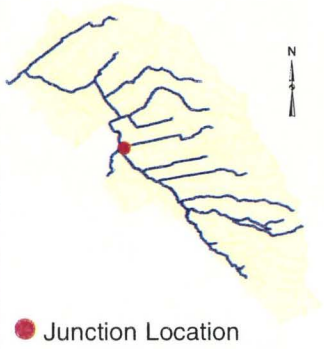


Figure 4-17
Percent Flow Increase at Location J-12



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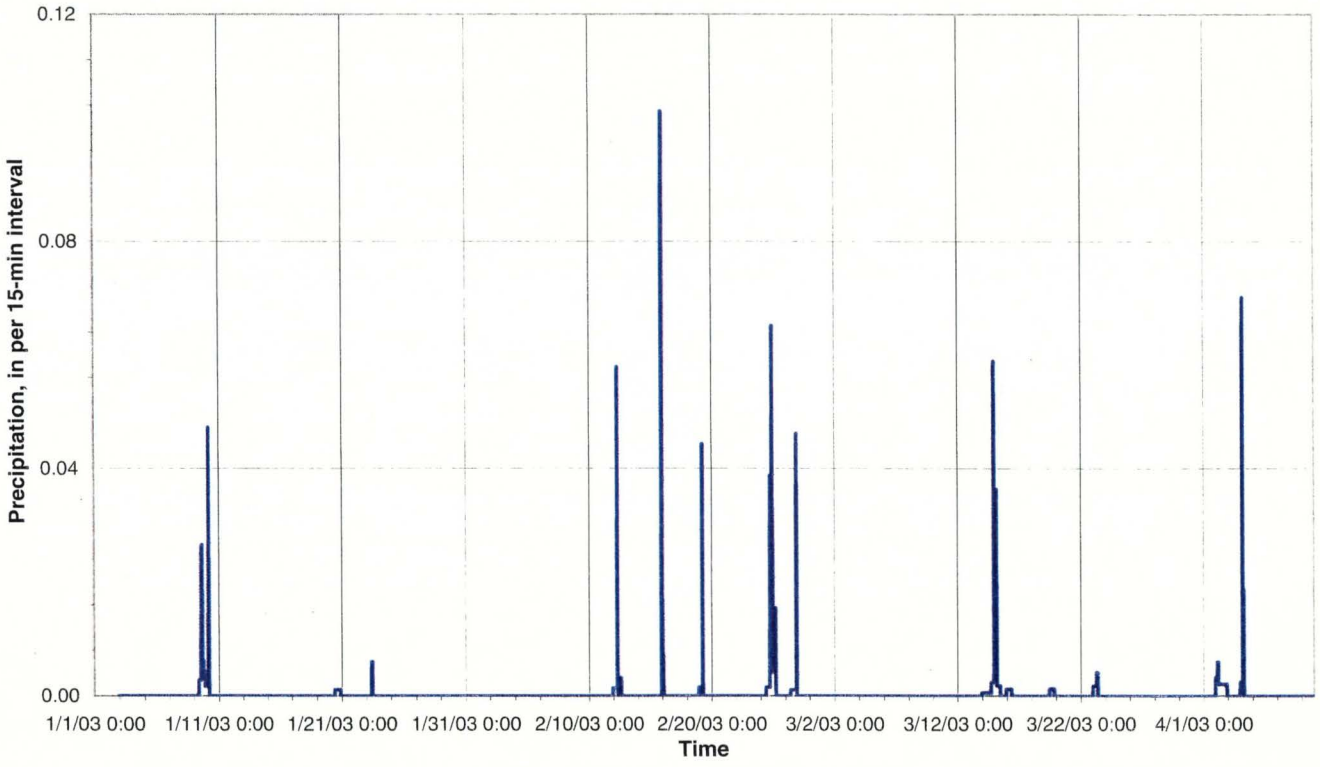


Figure 4-18
Spring 2003 Precipitation

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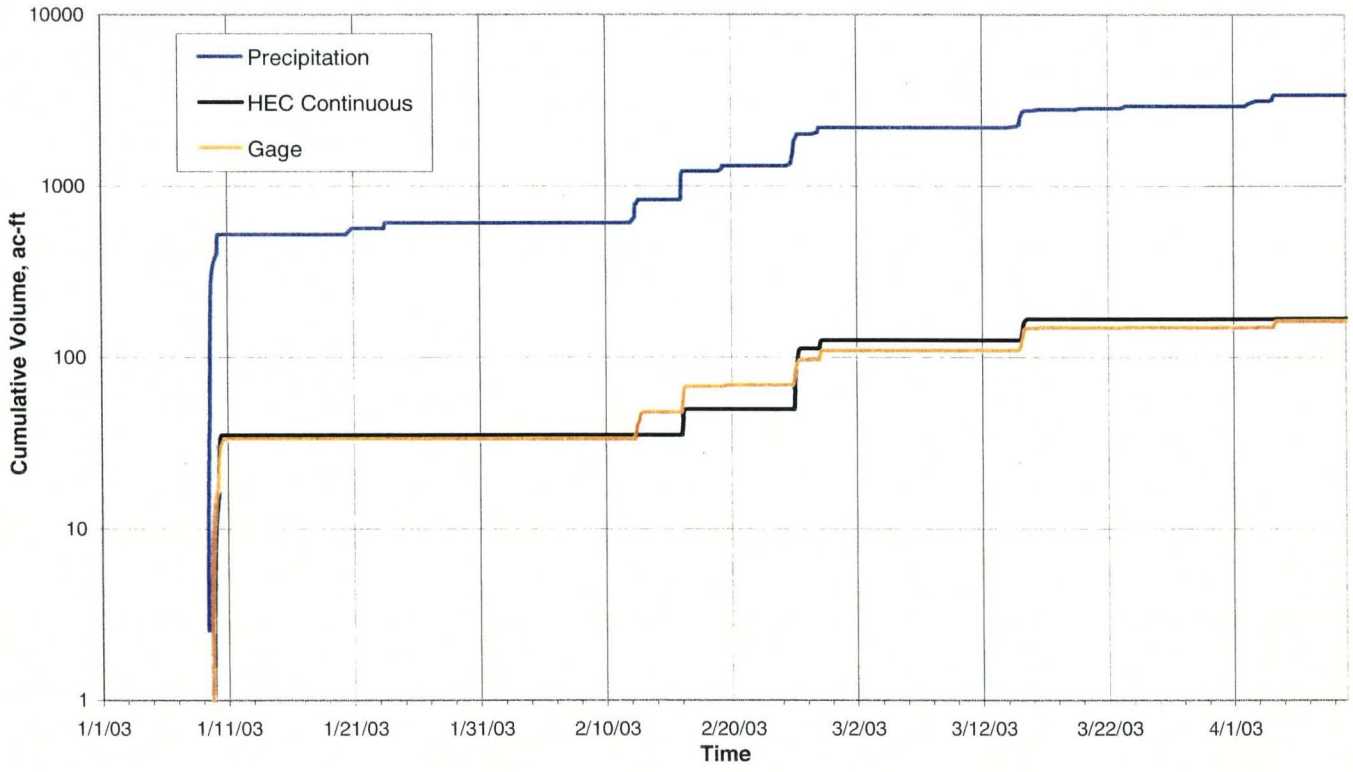


Figure 4-19
Cumulative Volume of Discharge -
Thompson Creek at Quimby Road

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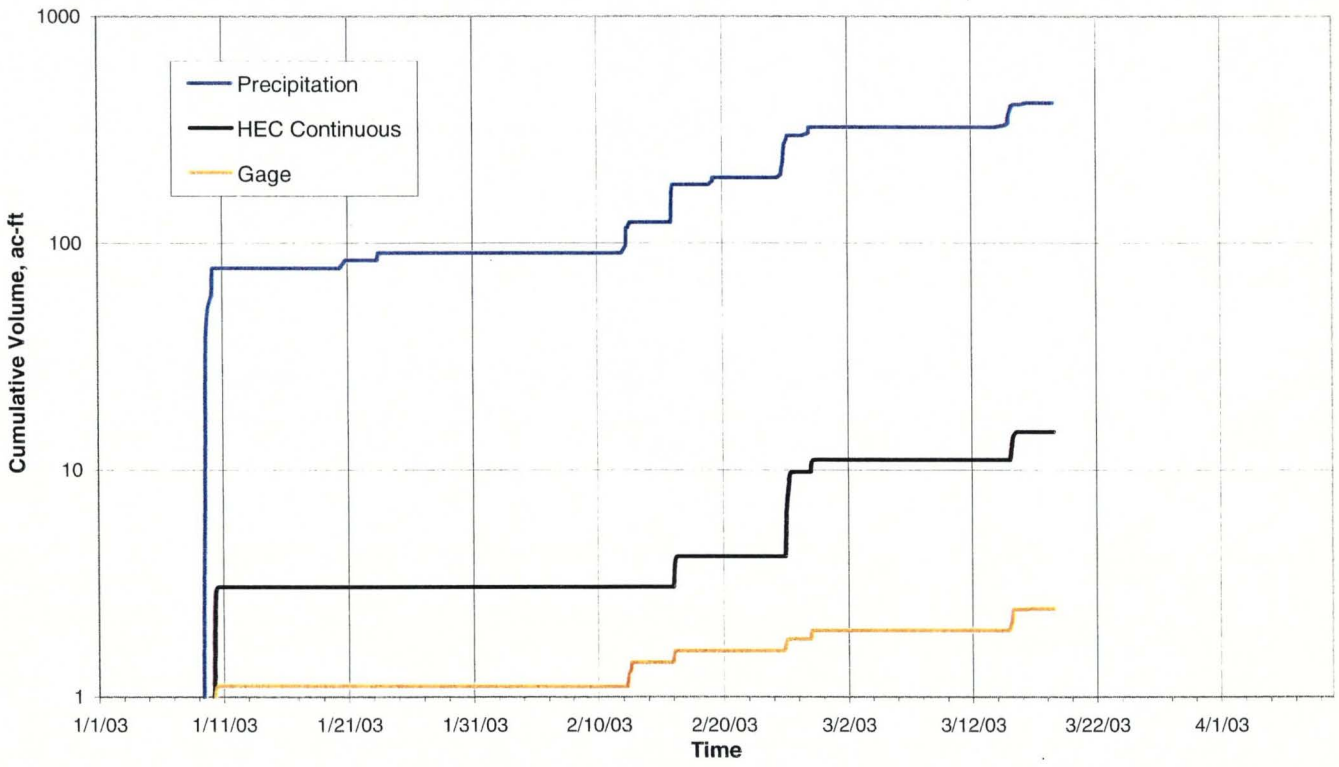


Figure 4-20
Cumulative Volume of Discharge -
Yerba Buena Creek Watershed

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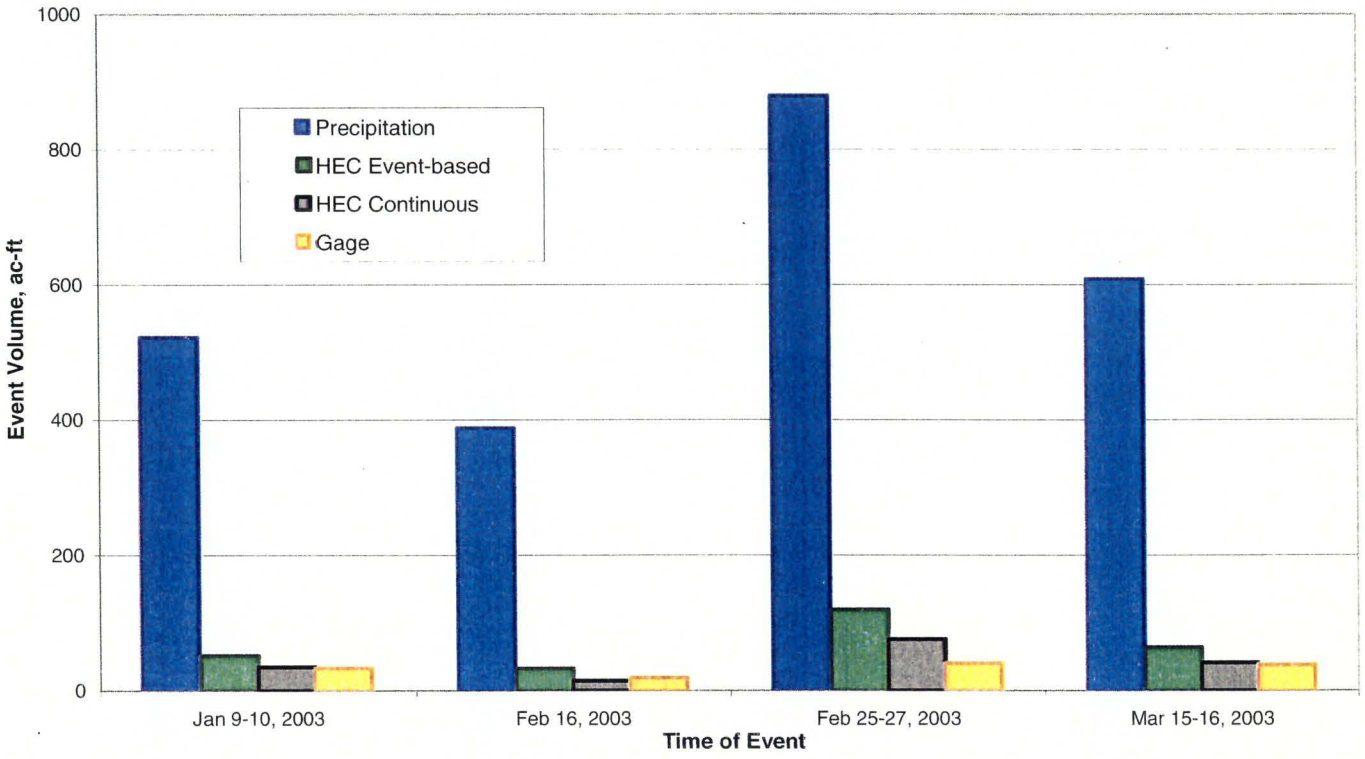


Figure 4-21
Event Volume Comparisons-
Thompson Creek at Quimby Road

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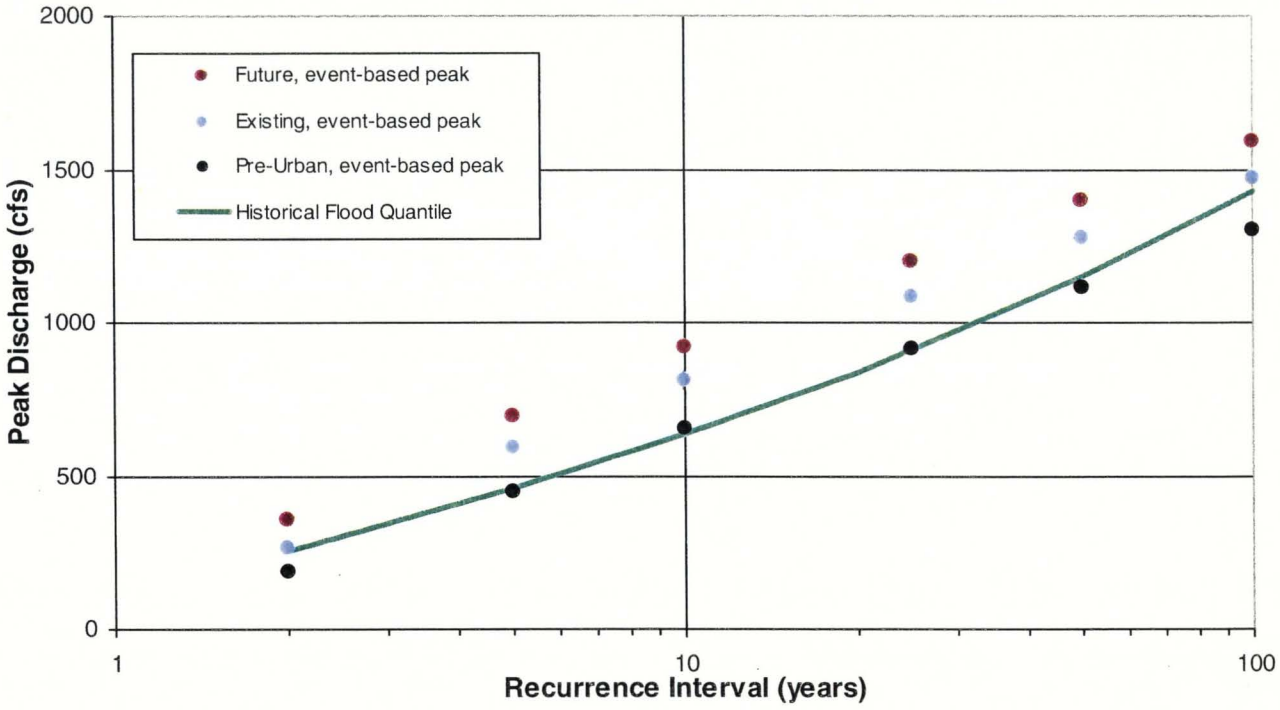
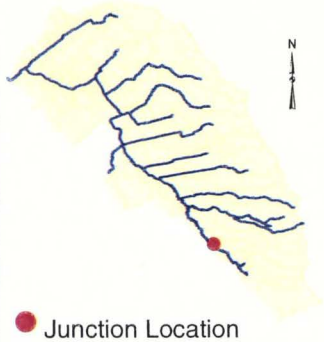


Figure 4-23
Thompson Creek Peak Discharge Recurrence Intervals, Event-Based Model at Junction J-5



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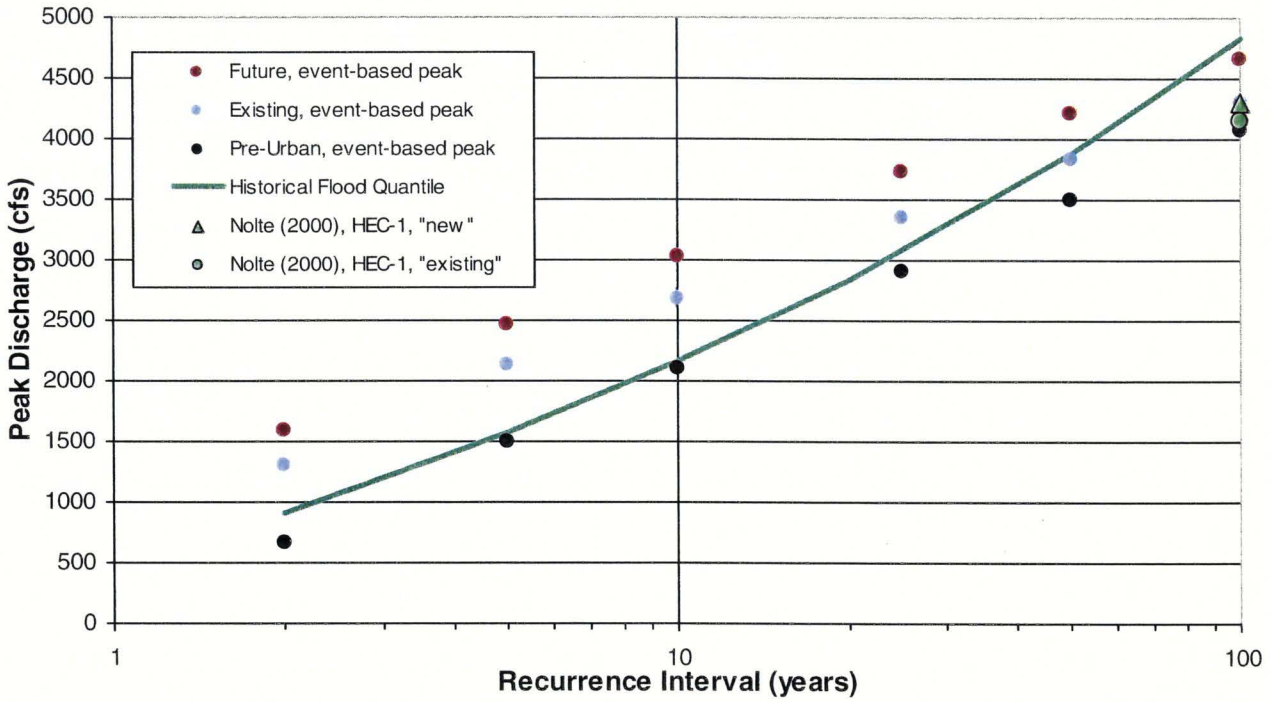
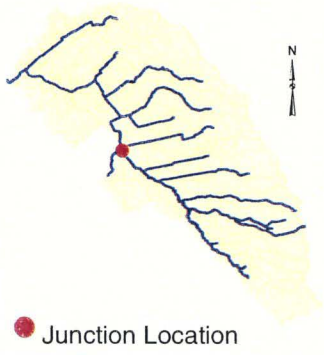


Figure 4-24
Thompson Creek Peak Discharge Recurrence Intervals, Event-Based Model at Junction J-12



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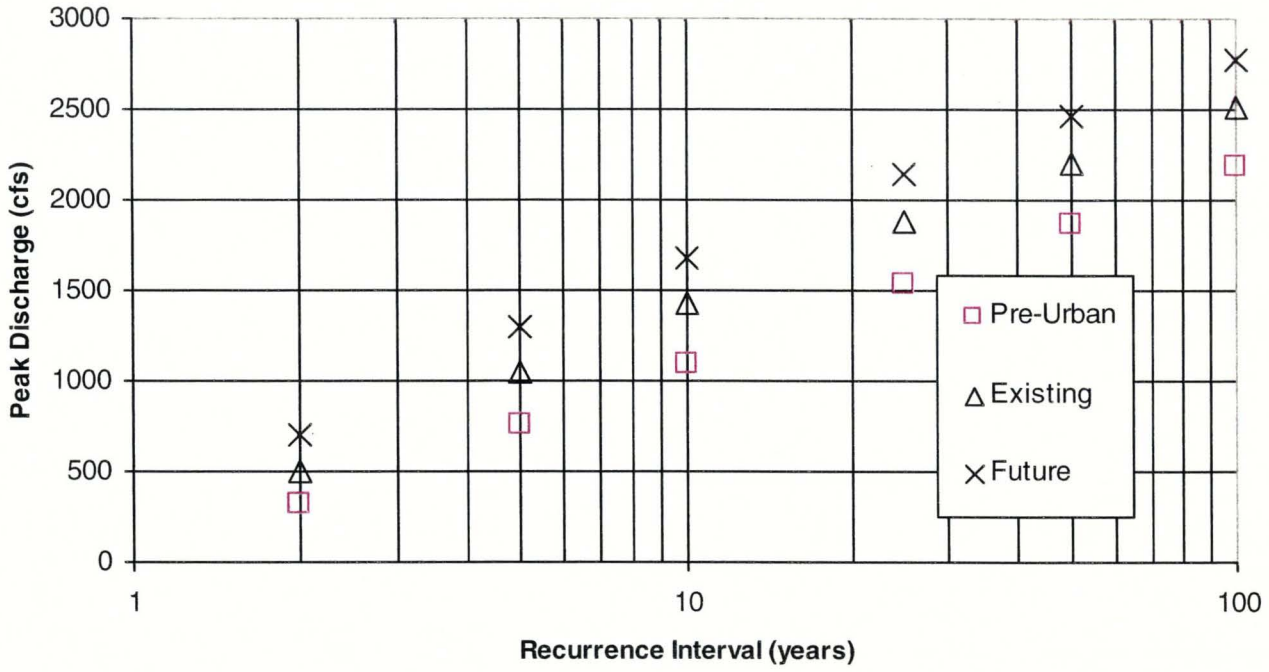
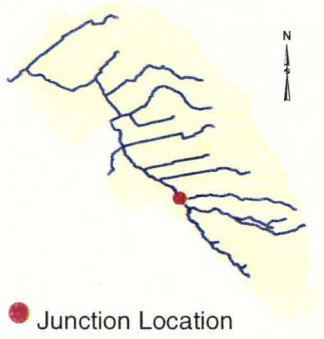


Figure 4-25
Thompson Creek Flood Frequency for Junction J-9 Using Event-Based Model Results



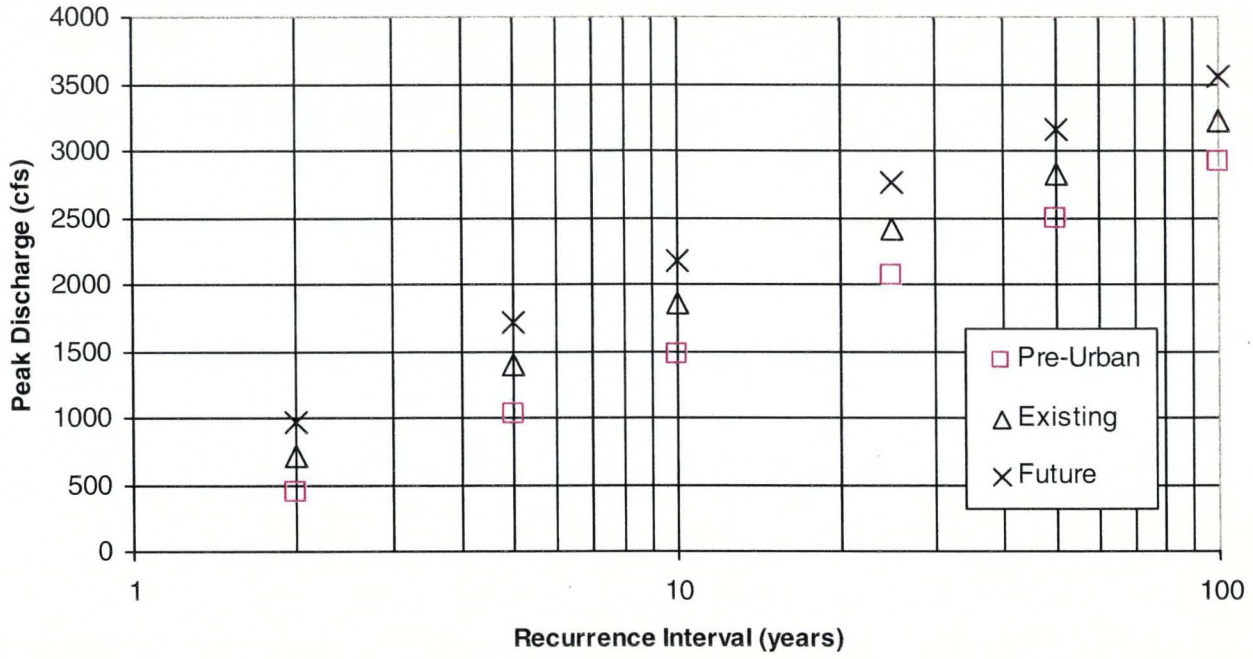


Figure 4-26
Thompson Creek Flood Frequency for Junction J-10 Using Event-Based Model Results



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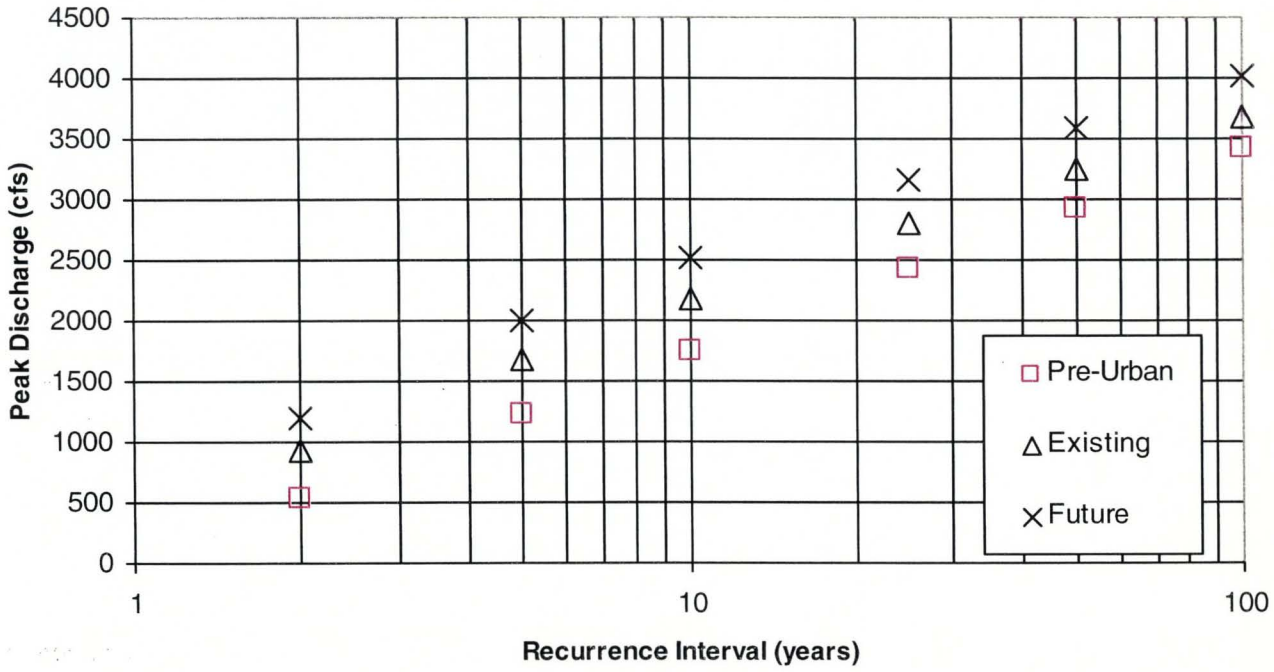


Figure 4-27
Thompson Creek Flood Frequency for Junction J-11 Using Event-Based Model Results



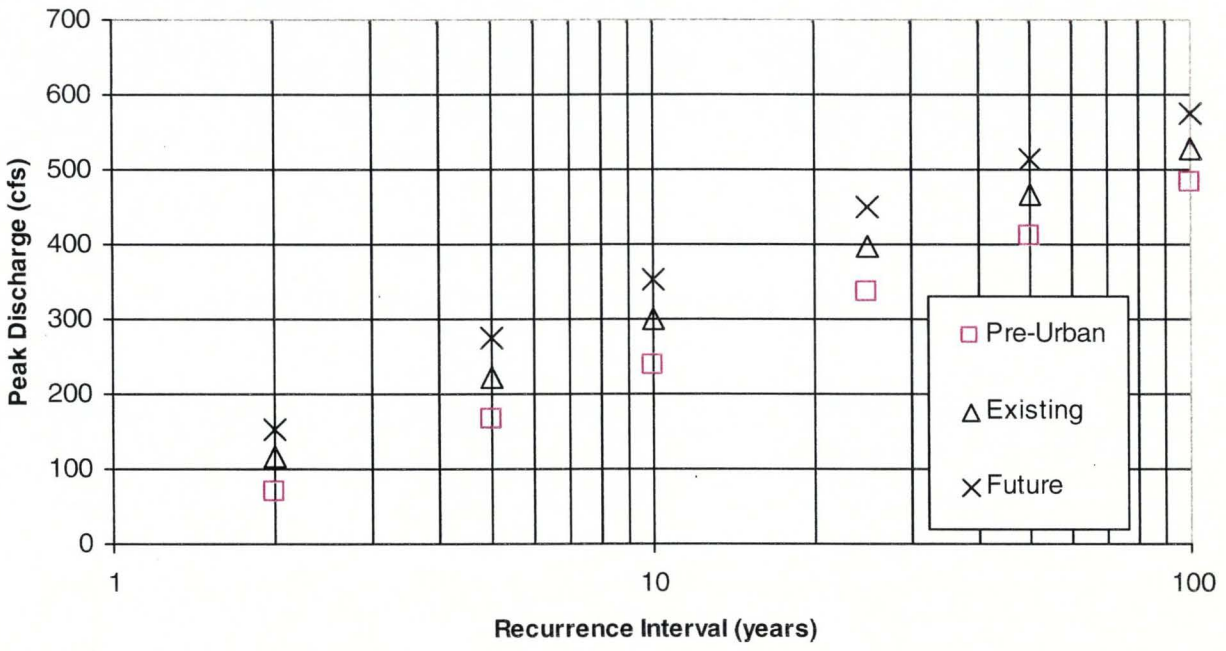


Figure 4-28
Thompson Creek Flood Frequency
at Cross-Section YB-7
Using Event-Based Model Results
(Results are inclusive of flows upstream
of YB-7 on Yerba Buena)



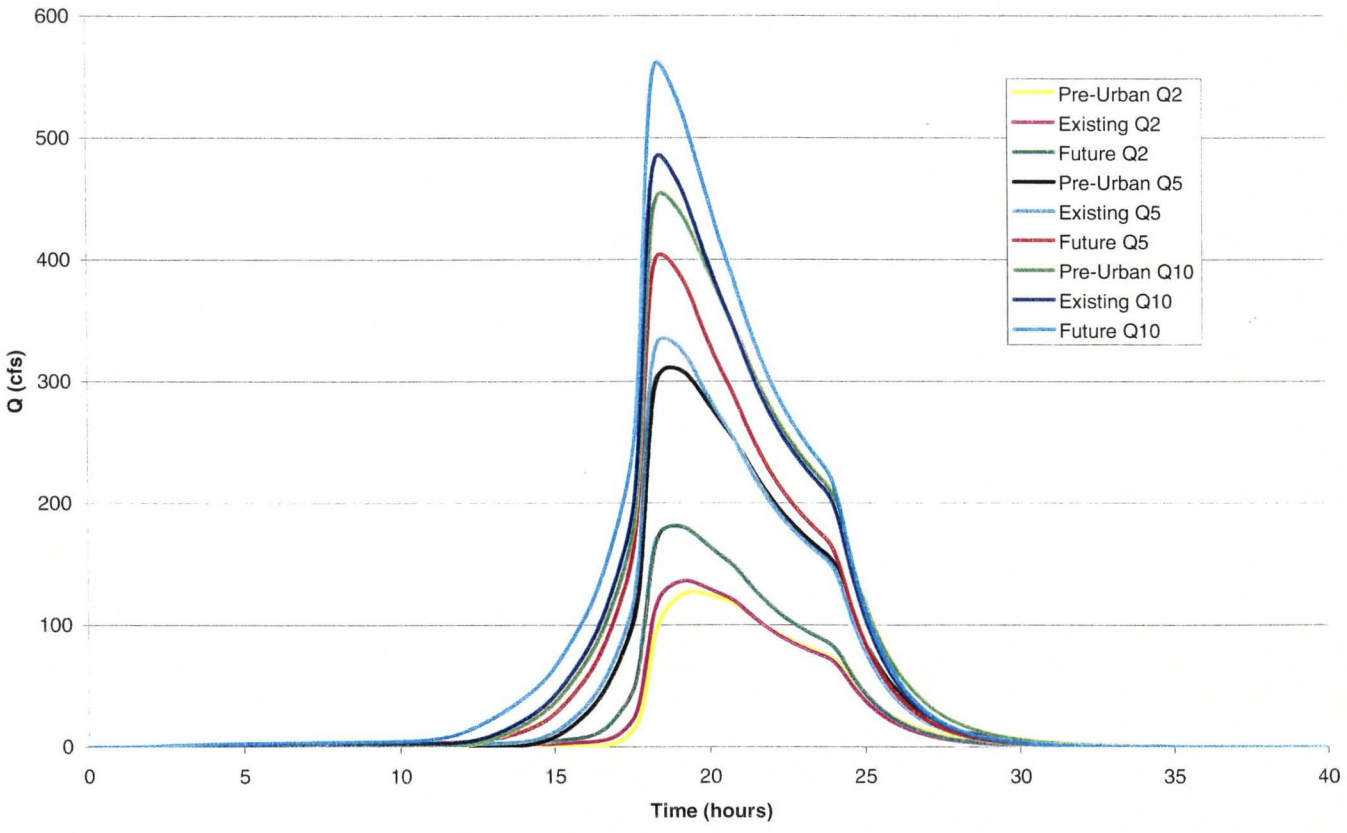


Figure 4-29
Thompson Creek Event-Based Storm Hydrographs at Junction J-3 for a 2-yr, 5-yr and 10-yr Event Using Pre-Urban, Existing and Future Land Use Conditions

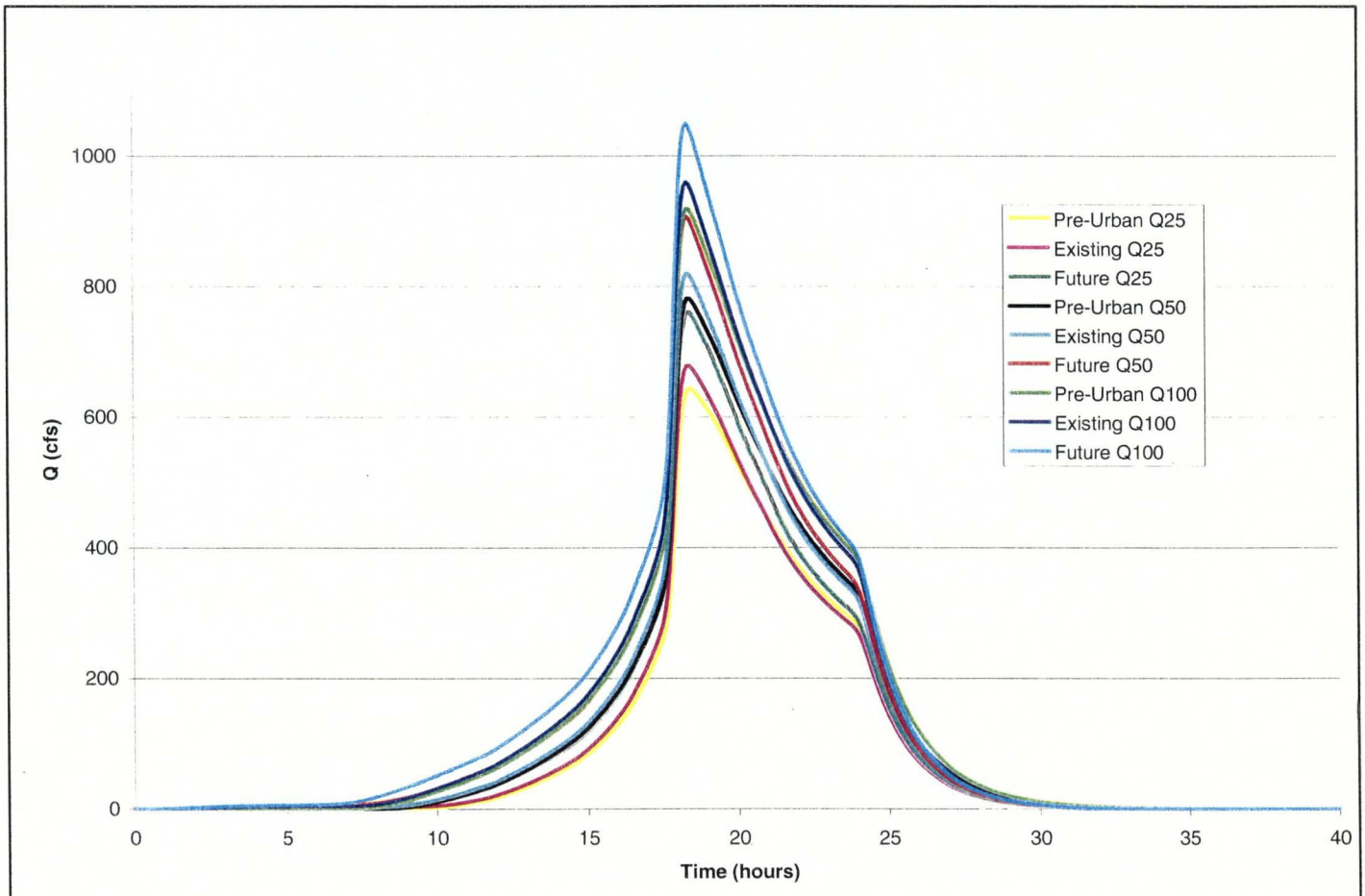


Figure 4-30
Thompson Creek Event-Based Storm Hydrographs at Junction J-3 for a 25-yr, 50-yr and 100-yr Event Using Pre-Urban, Existing and Future Land Use Conditions

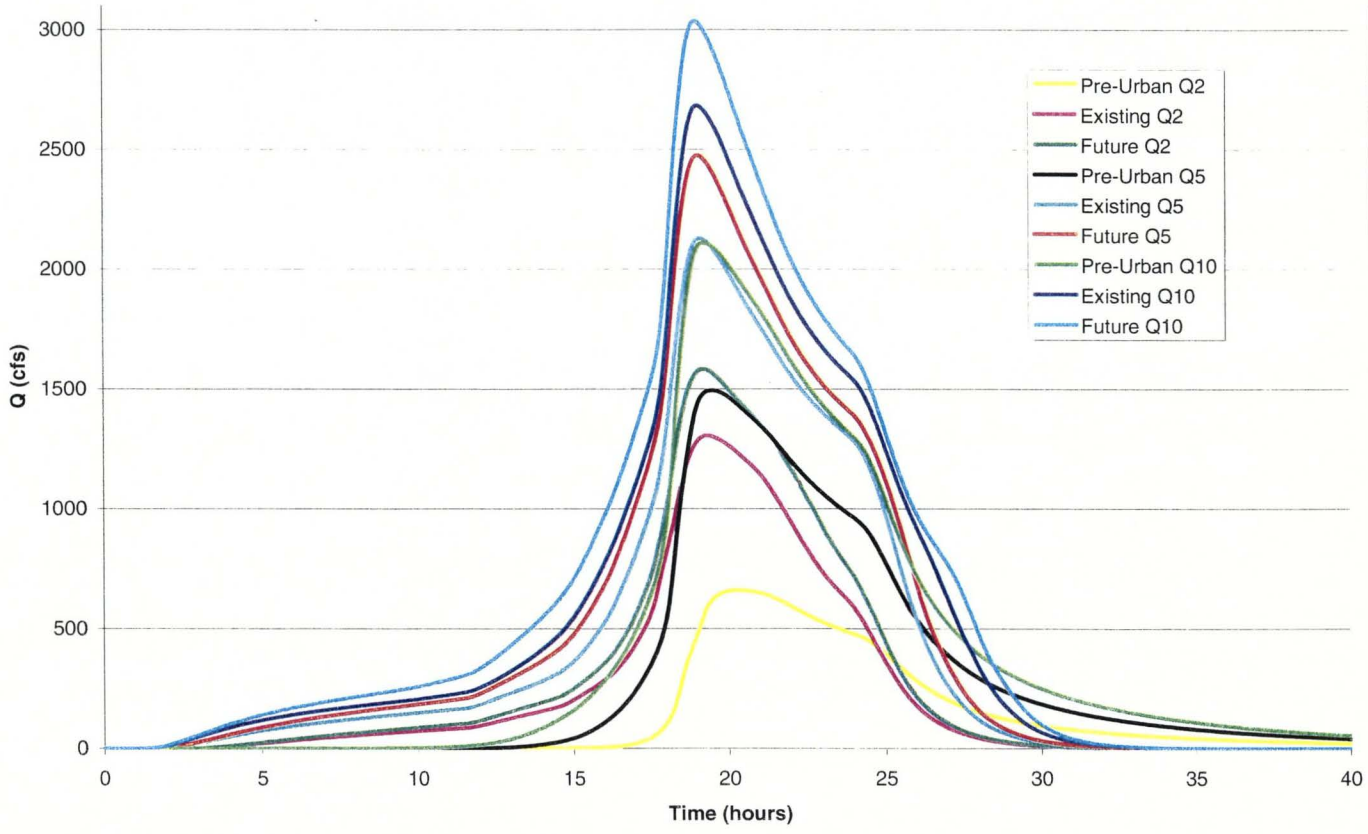


Figure 4-31
Thompson Creek Event-Based Storm Hydrographs at Junction J-12 for a 2-yr, 5-yr and 10-yr Event Using Pre-Urban, Existing and Future Land Use Conditions

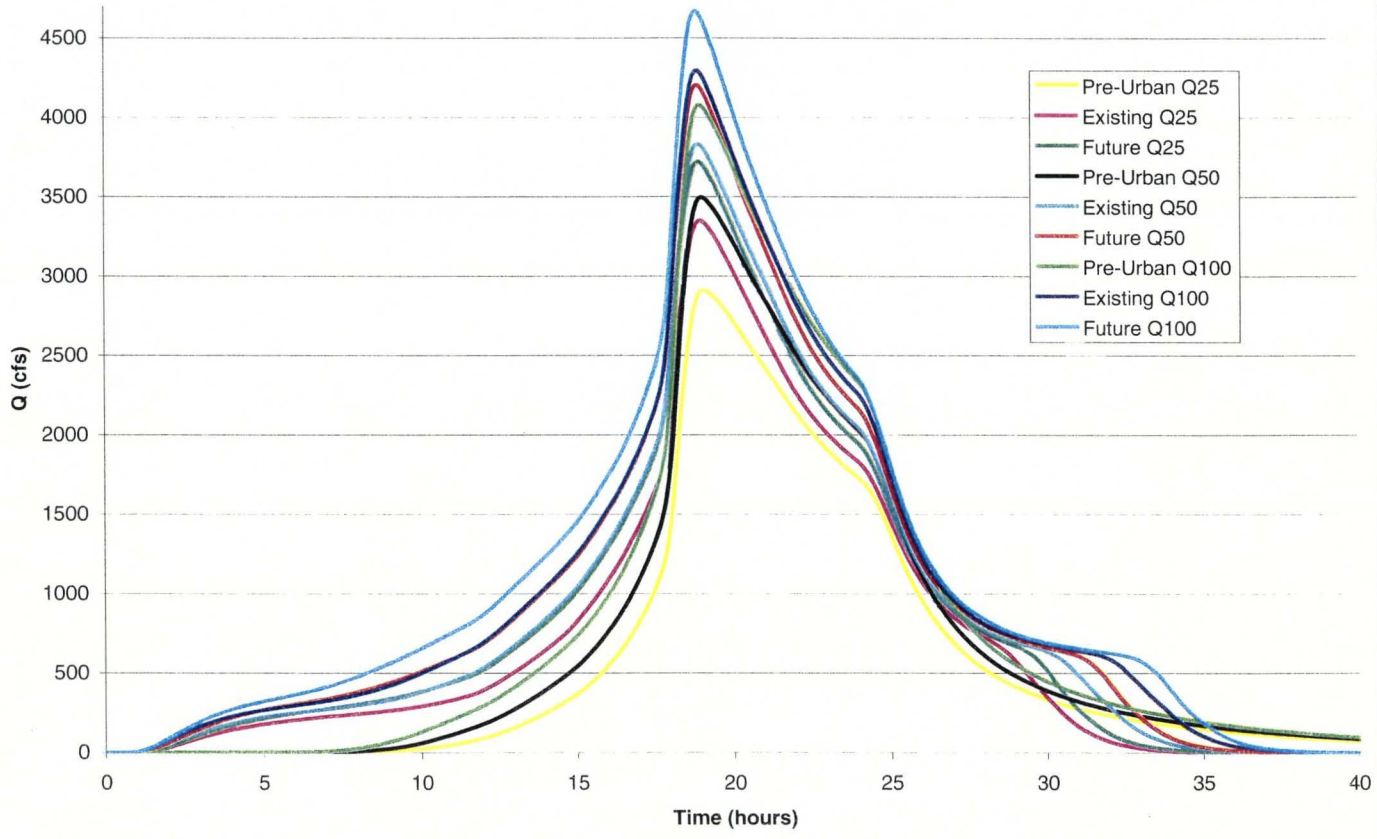


Figure 4-32
Thompson Creek Storm Hydrographs at Junction J-12 for a 25-yr, 50-yr and 100-yr Event Using Pre-Urban, Existing and Future Land Use Conditions

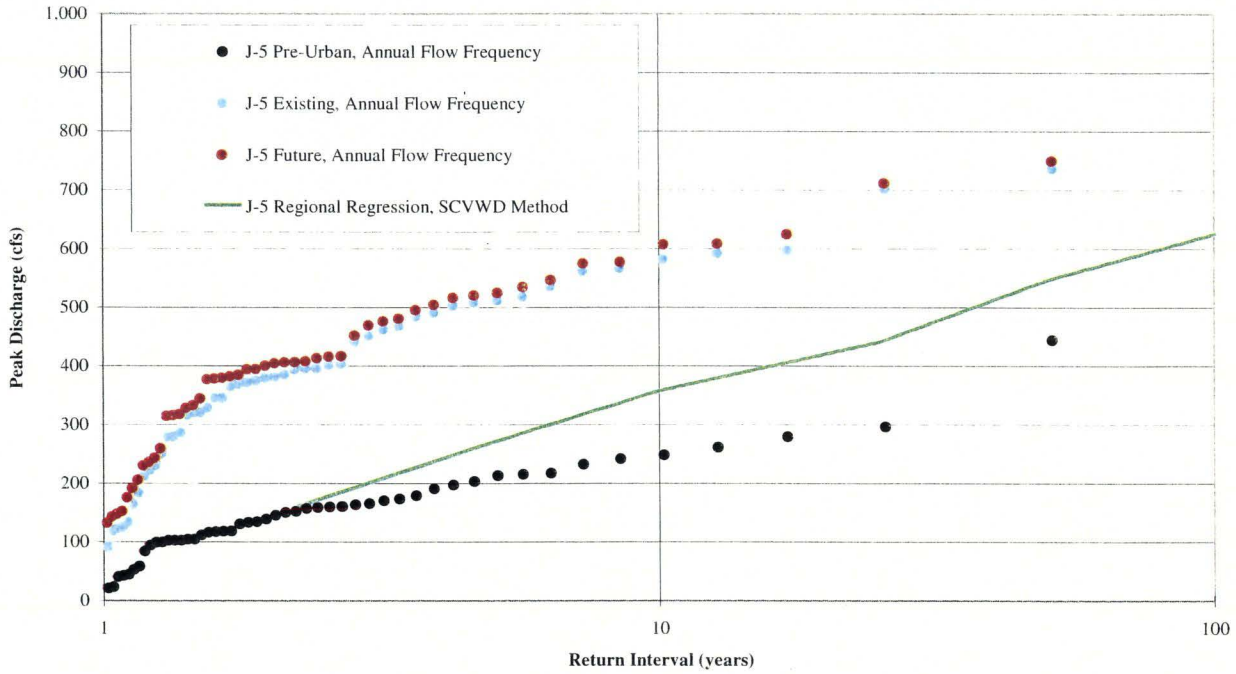
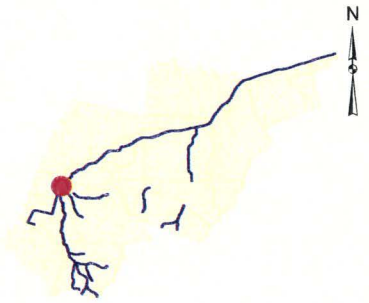


Figure 4-33
Flood Frequency for Ross Creek Junction J-5 Using Continuous Model Results



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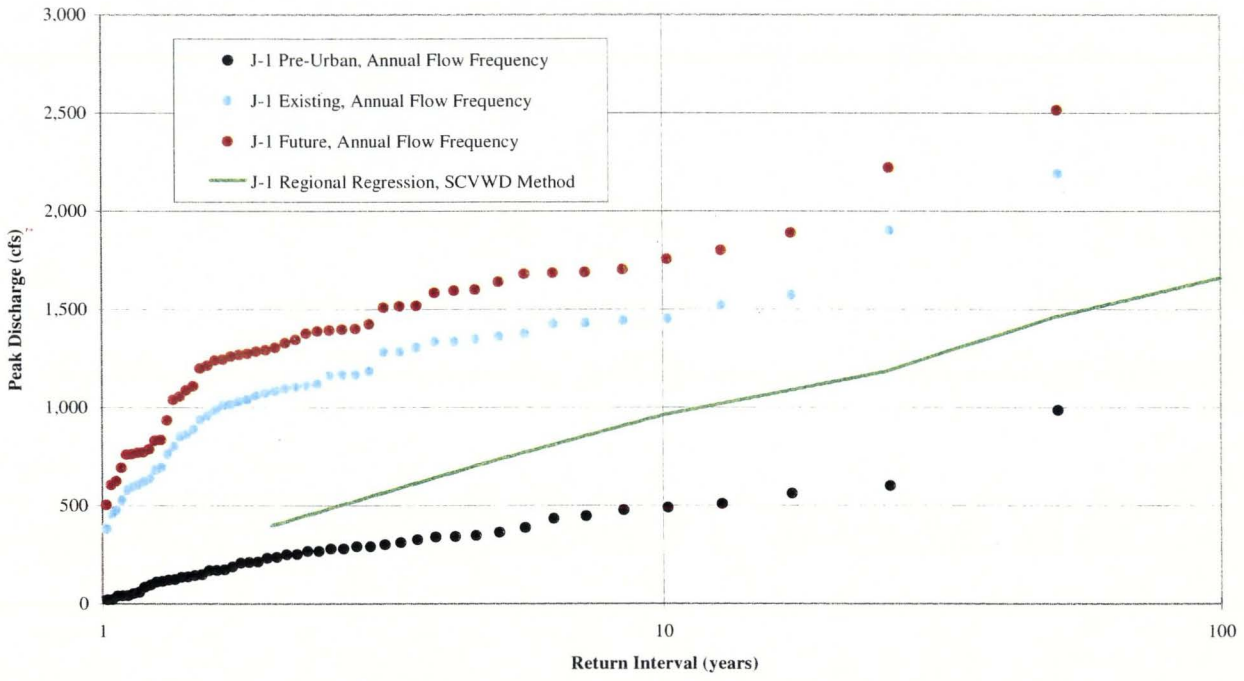
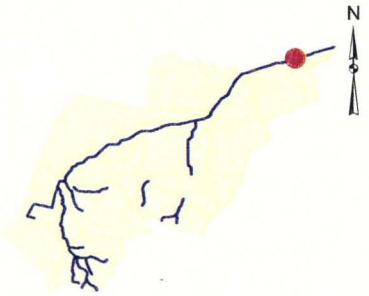


Figure 4-34
Flood Frequency for Ross Creek Junction J-1 Using Continuous Model Results



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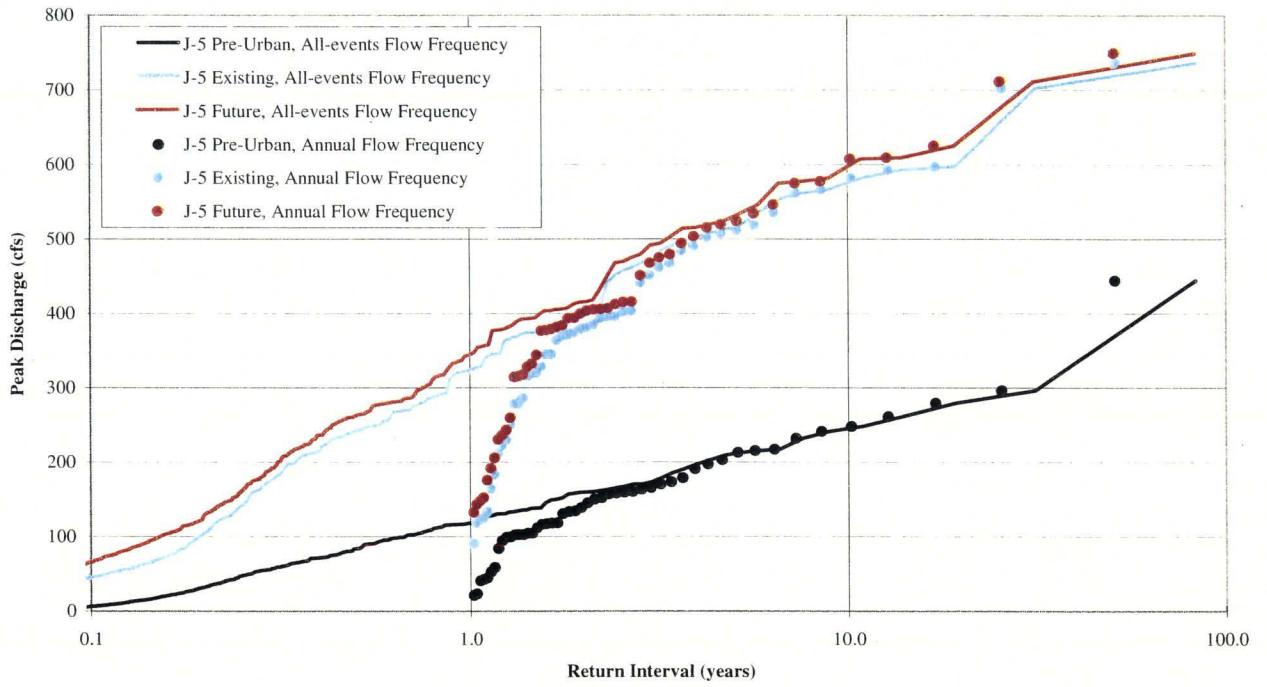


Figure 4-35
Ross Creek Peak Flow Frequency Analysis: Monthly and Annual Recurrence Intervals at Junction J-5



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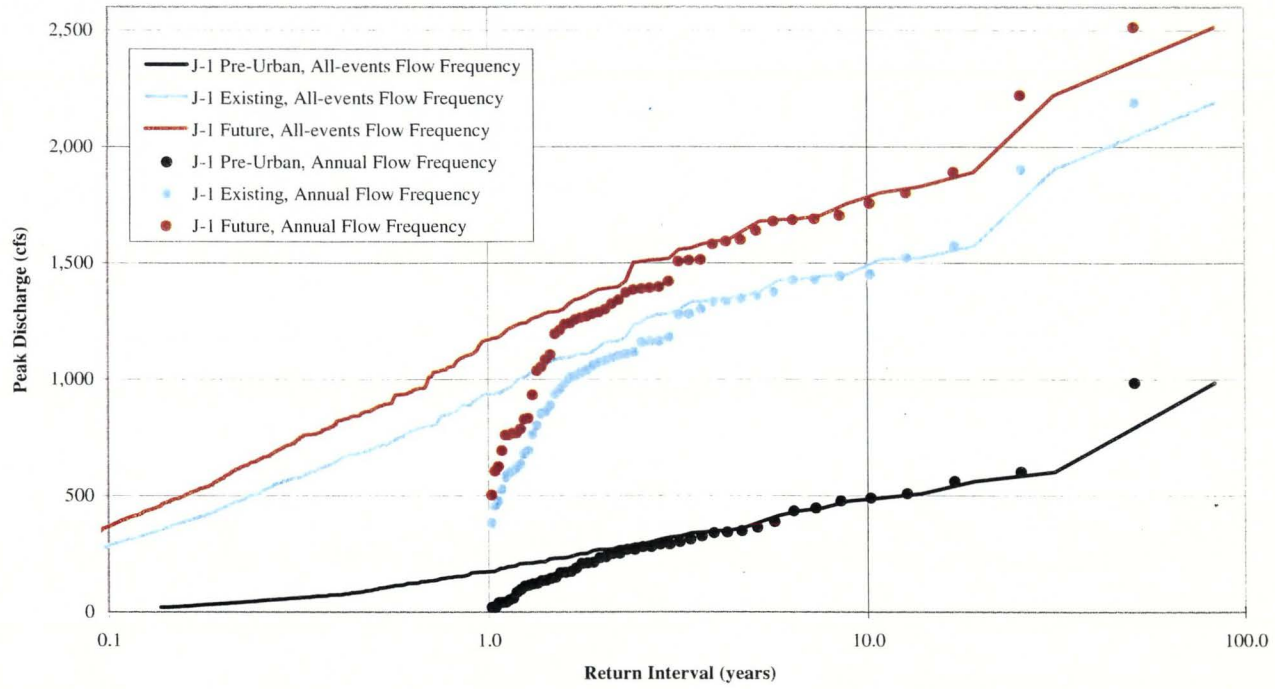
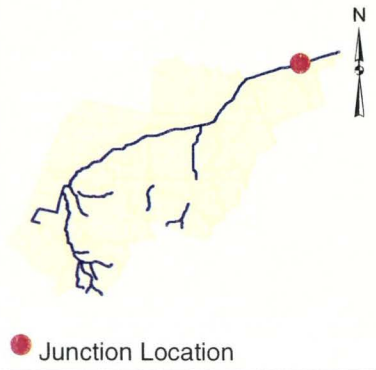


Figure 4-36
Ross Creek Peak Flow Frequency Analysis: Monthly and Annual Recurrence Intervals at Location J-1



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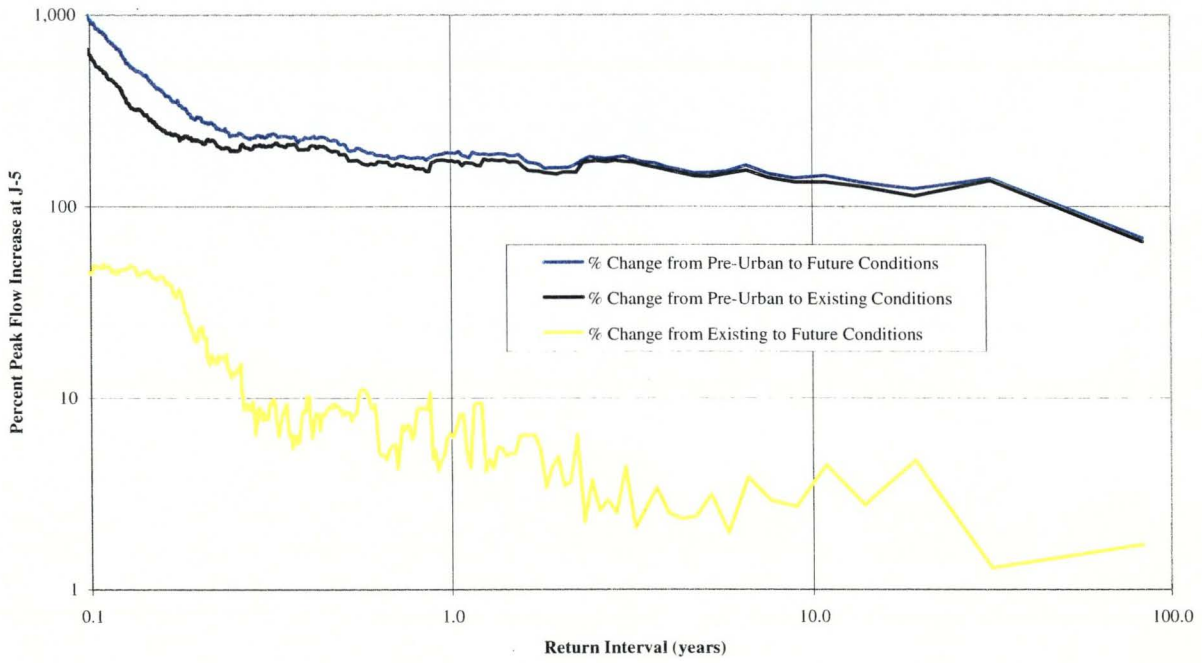
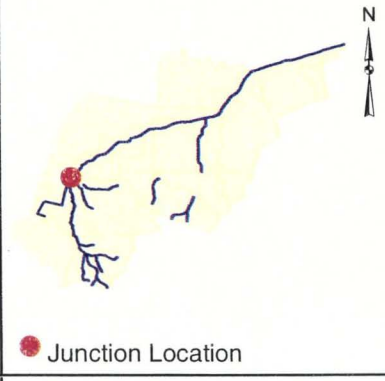


Figure 4-37
Percent Flow Increase at Ross Creek at Junction J-5



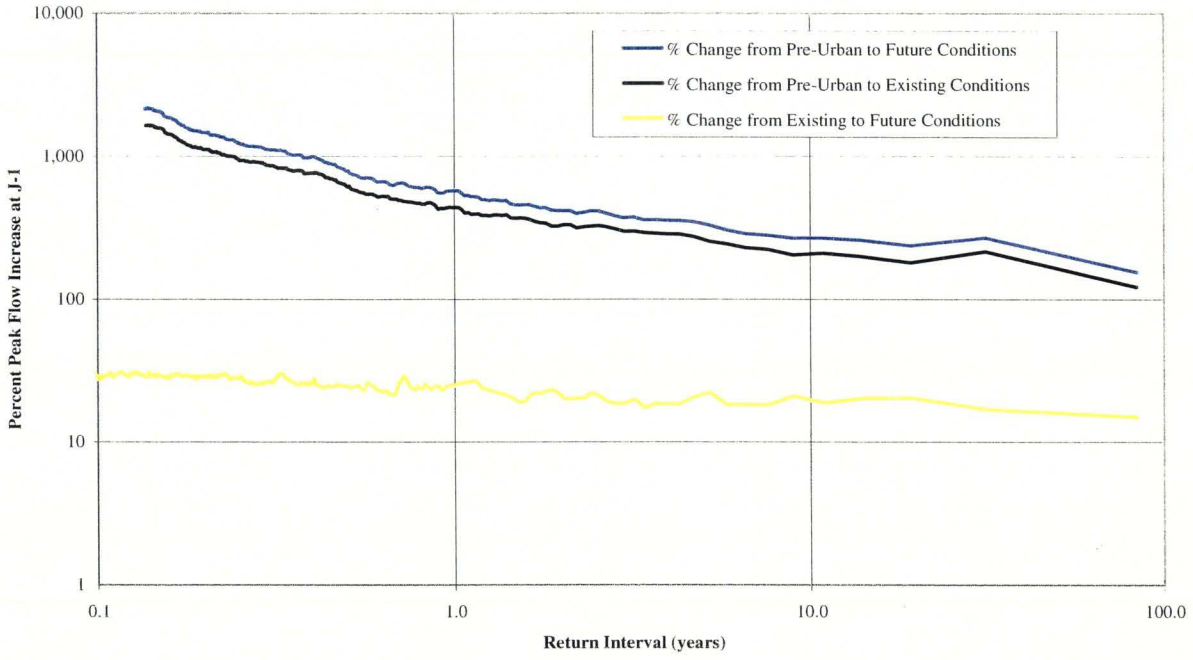
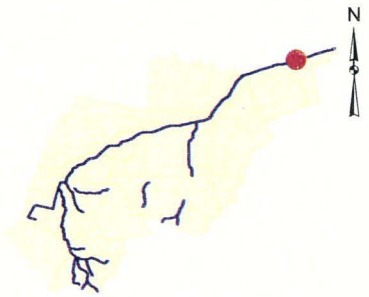


Figure 4-38
Percent Flow Increase at Ross Creek Junction J-1



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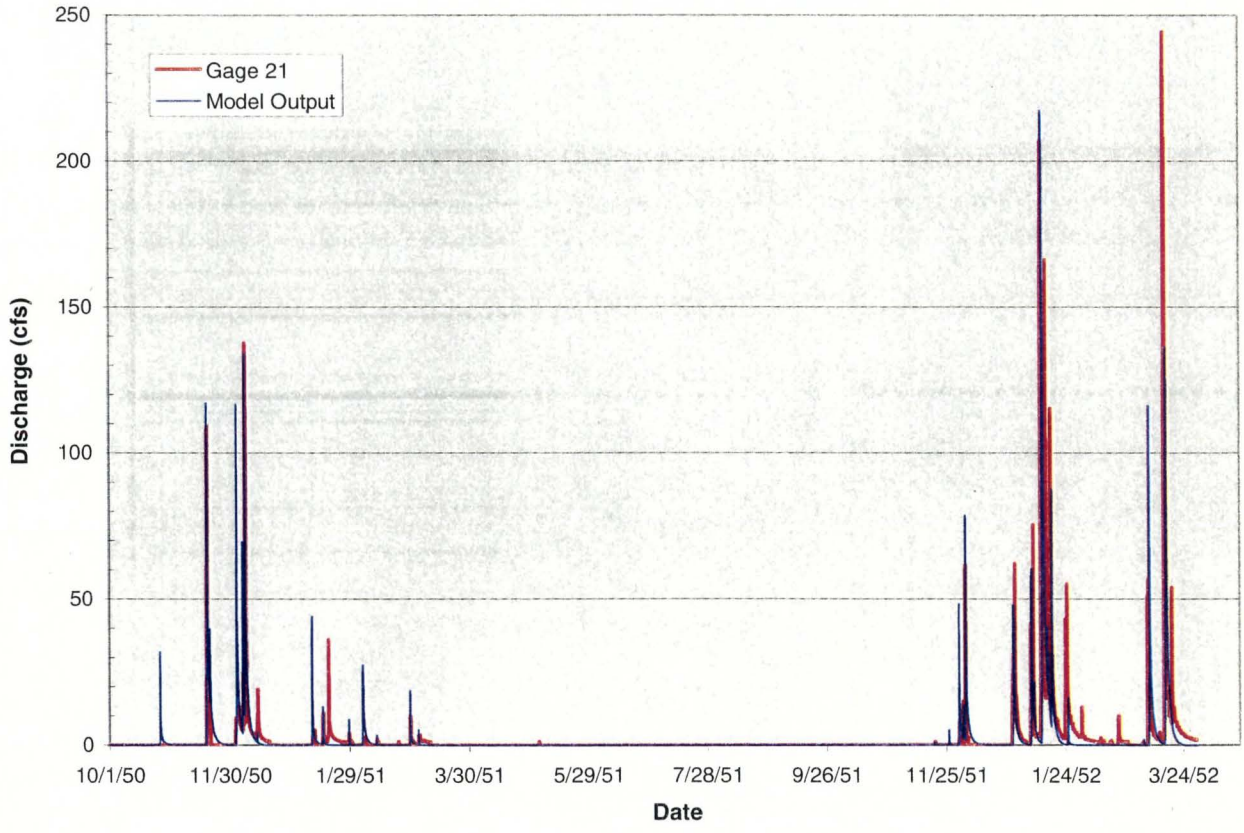


Figure 4-39
Pre-Development Calibration Hydrographs for Gage 21 and Model Junction J-5

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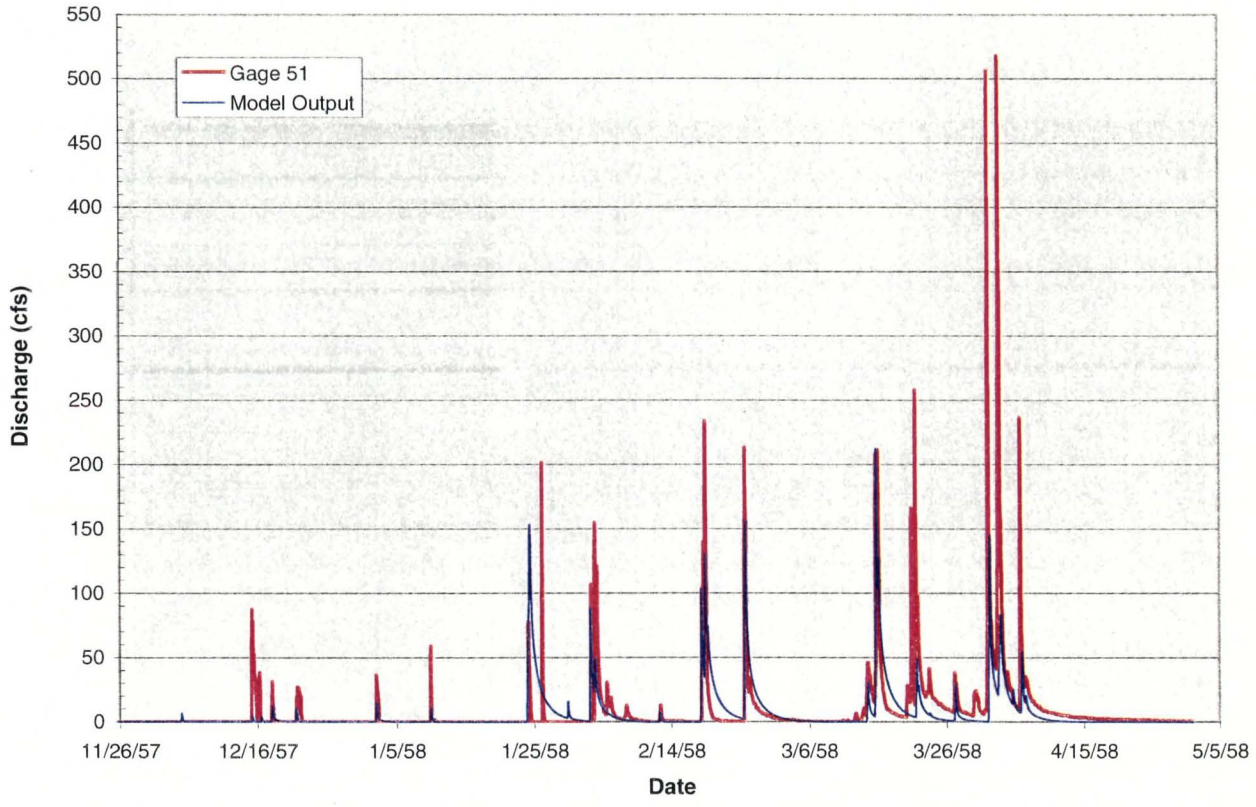


Figure 4-40
Pre-Development Calibration Hydrographs for Gage 51 and Model Junction J-1

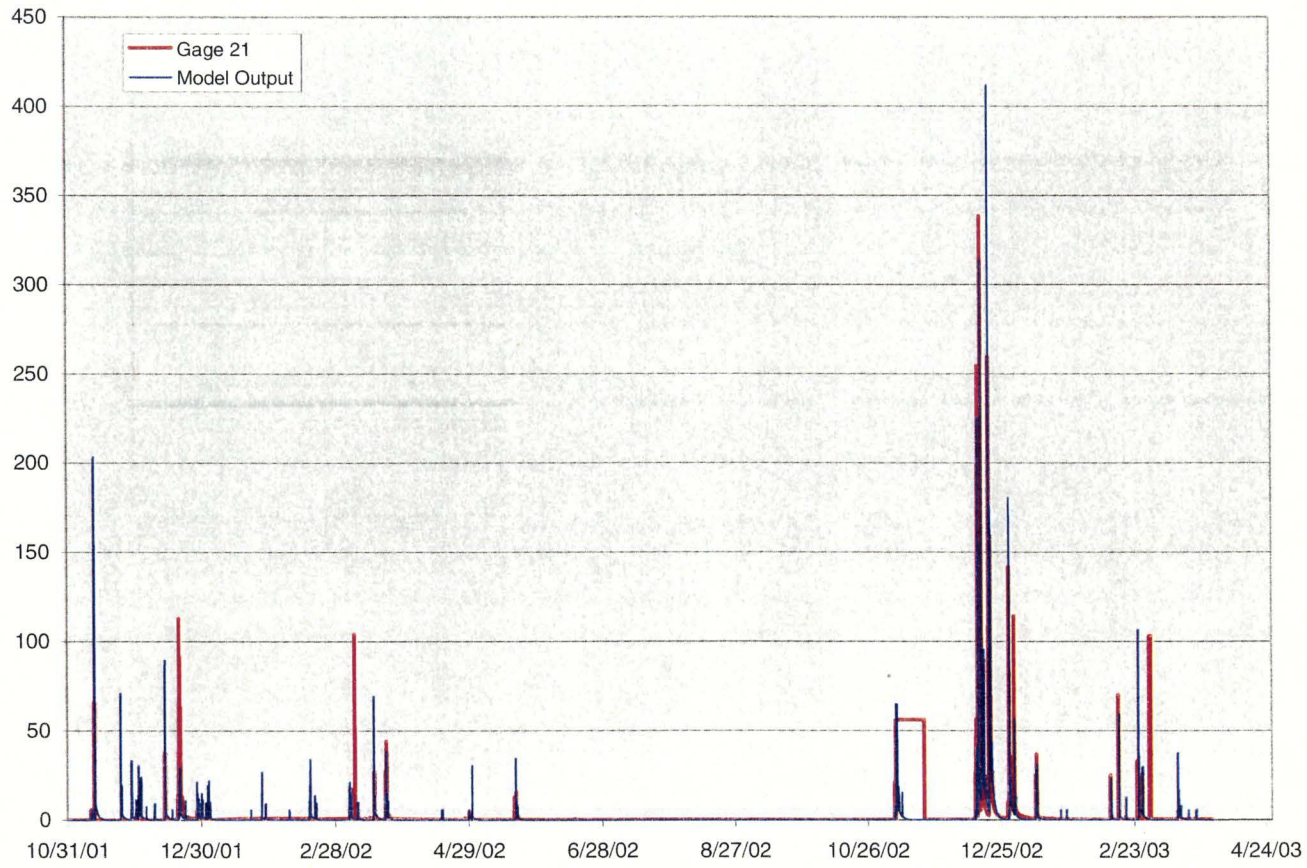


Figure 4-41
Existing Condition Calibration Hydrographs for Gage 21 and Model Junction J-5

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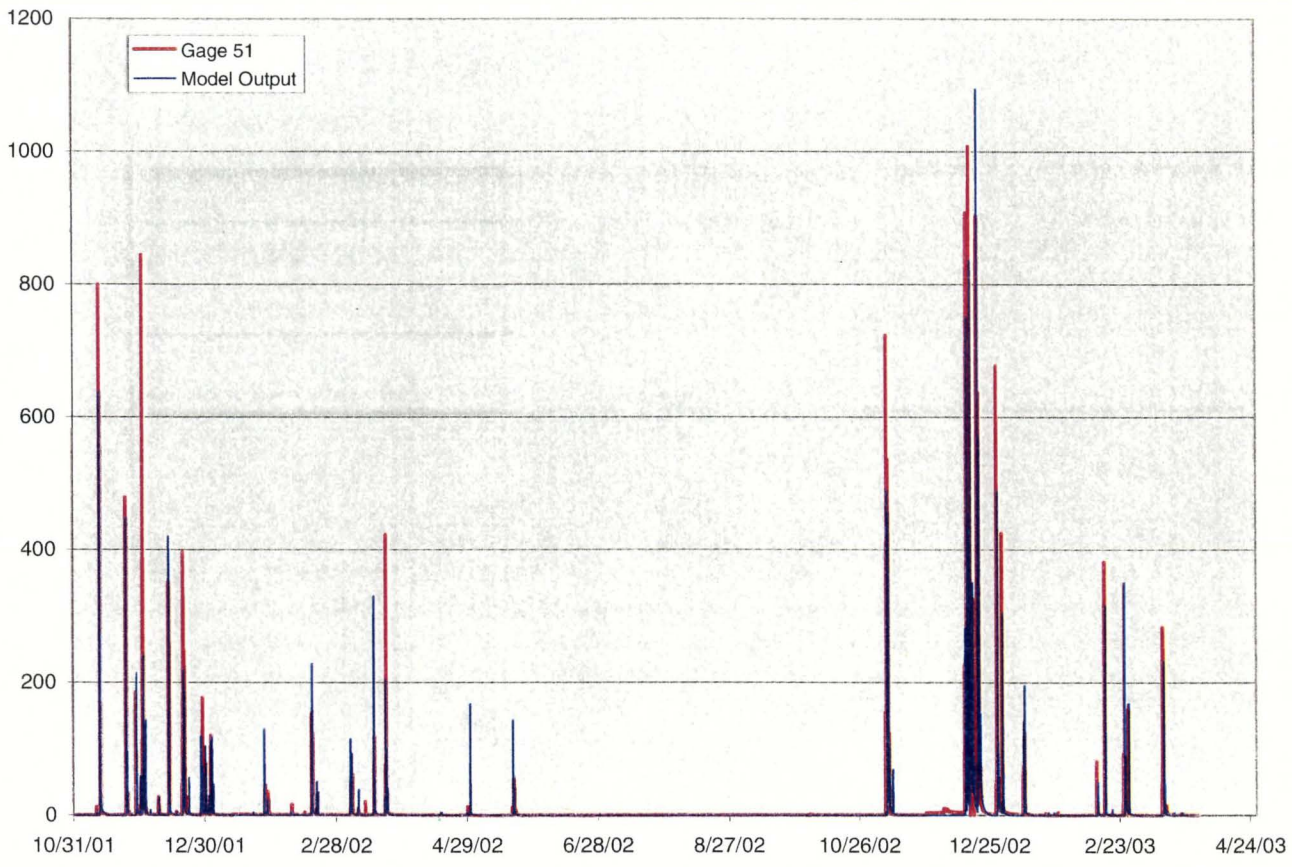
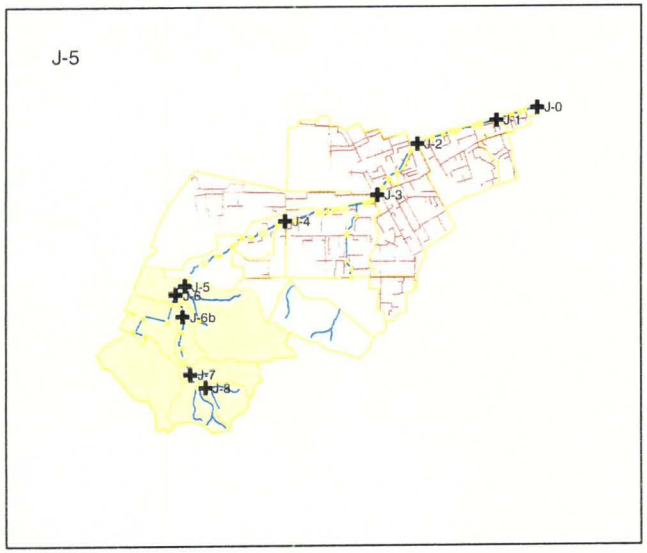
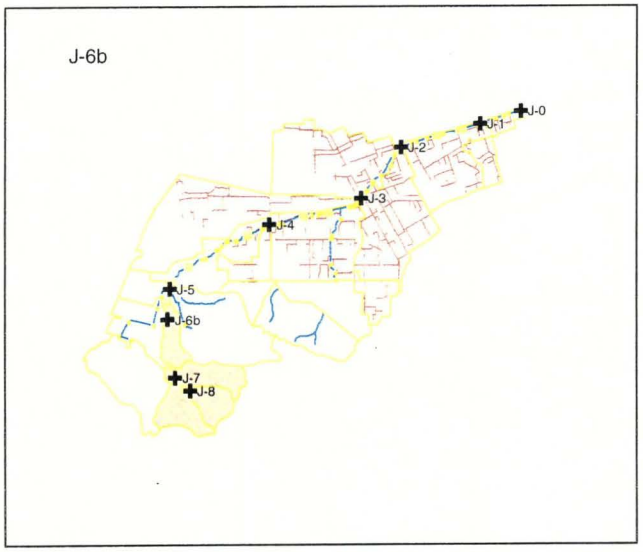
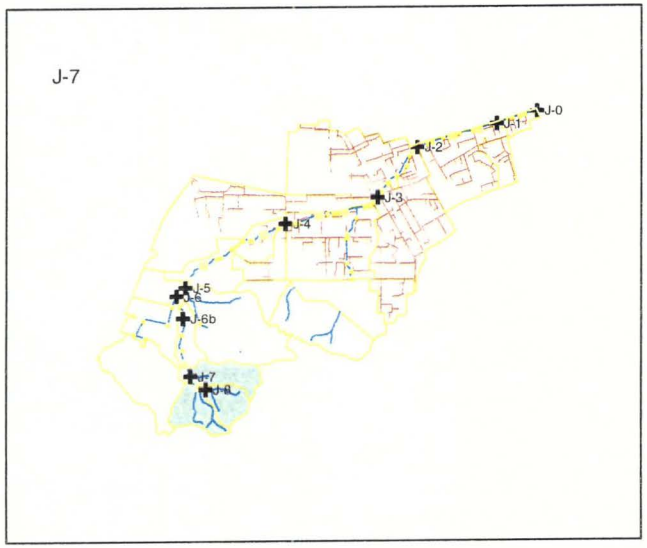


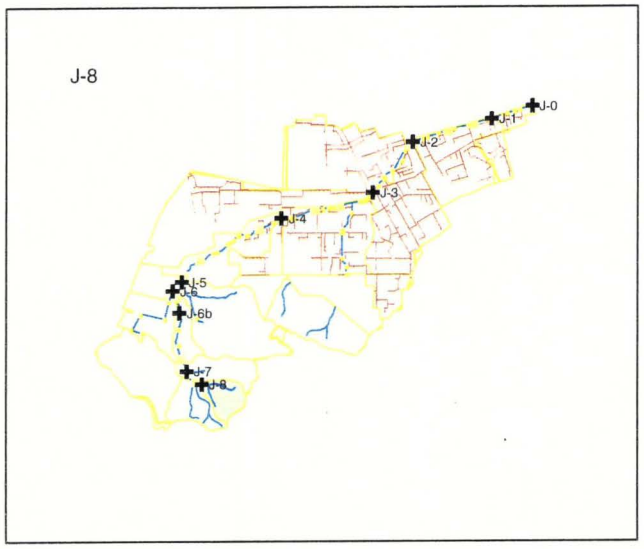
Figure 4-42
Existing Condition Calibration Hydrographs for Gage 51 and Model Junction J-1

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Cross-Section Name	Area*	Area IMP	Projected Area IMP	% IMP	Projected % IMP
J-5	1764.8	329.4792	631.0654	18.7	35.8
J-6b	540.1	48.1426	144.7229	8.9	26.8
J-7	417.6	24.3776	97.5604	5.8	23.4
J-8	115	1.38	20.7	1.2	18.0

* areas measured in acres

Legend

- Junction
- Cross-Section
- Outfall
- Catchment Boundary
- Creek
- Mains & Laterals

N

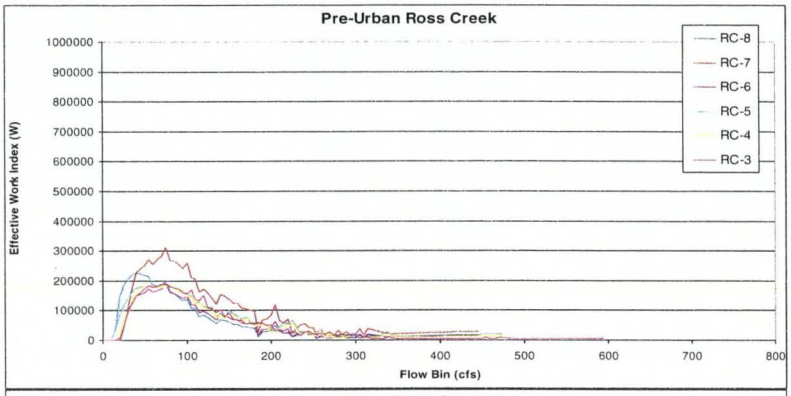
0 — 7,500 Feet

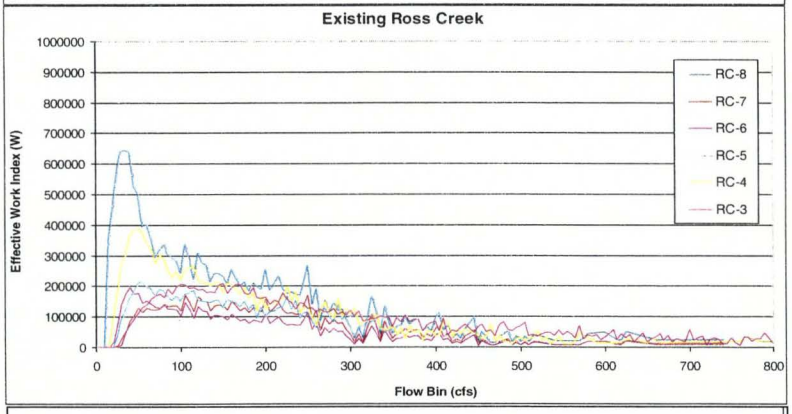
Figure Number 5-1
Areas and Percent Imperviousness for Sub-basins Draining into Cross-Sections in Ross Creek Junctions

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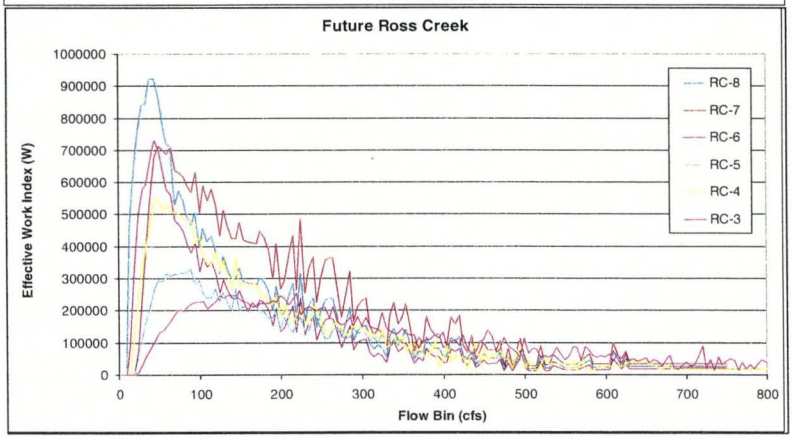
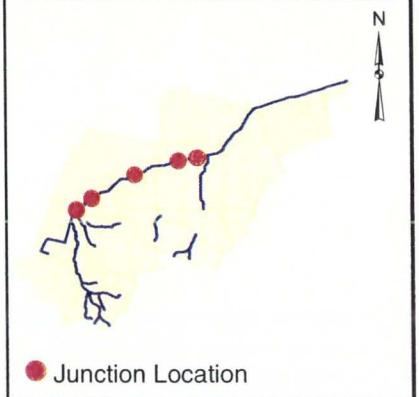


Figure 5-2
Effective Work Curves for Pre-urban, Existing, and Future Watershed Conditions for Ross Creek

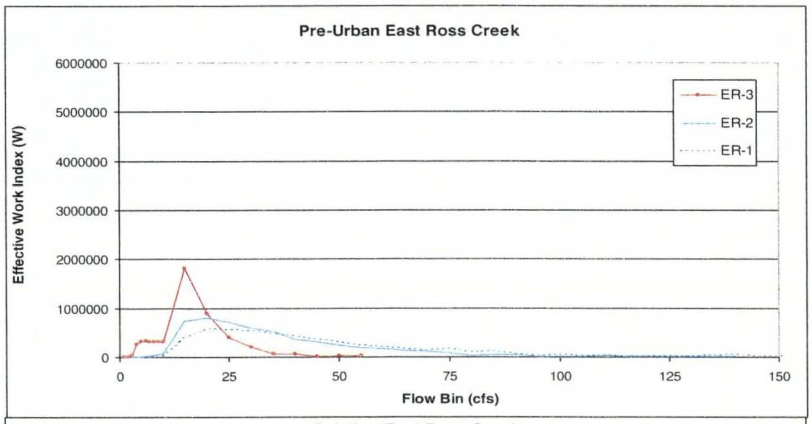


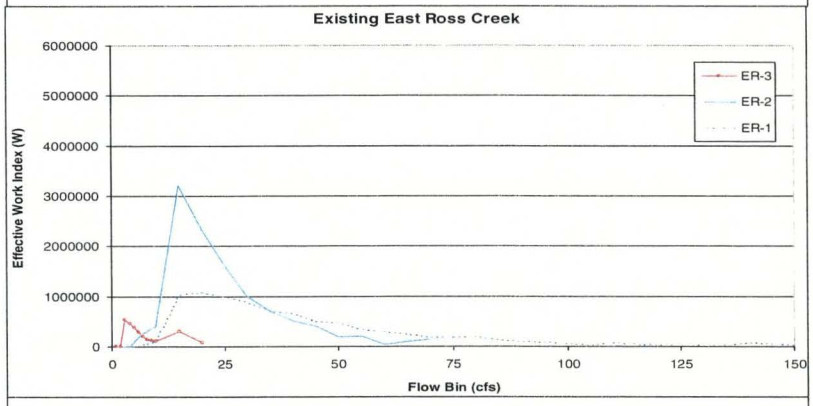
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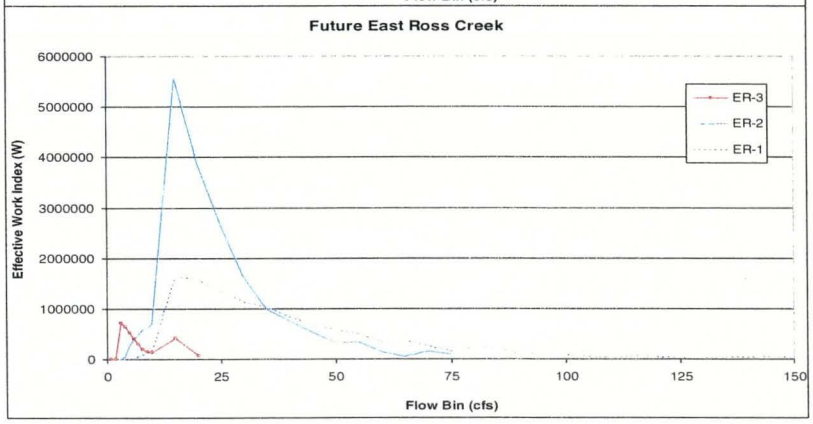
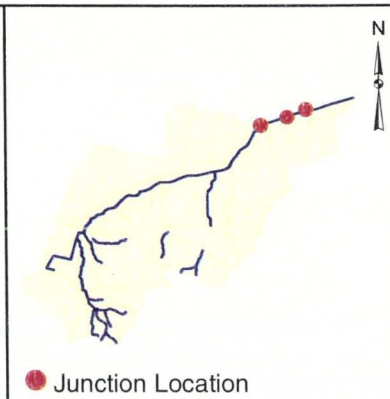


Figure 5-3
Effective Work Curves for Pre-urban, Existing, and Future Watershed Conditions for East Ross Creek

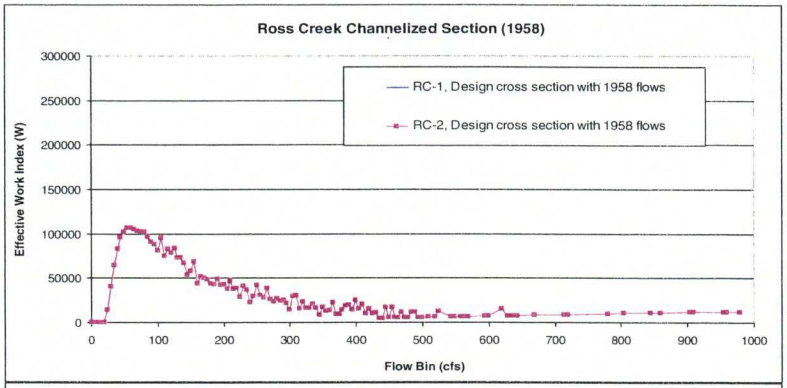


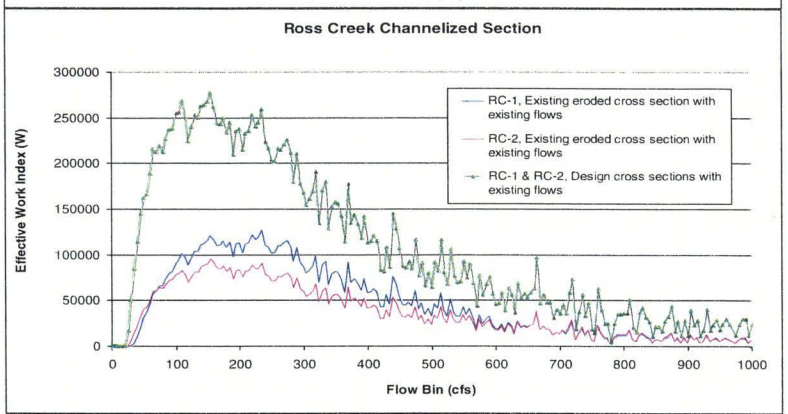
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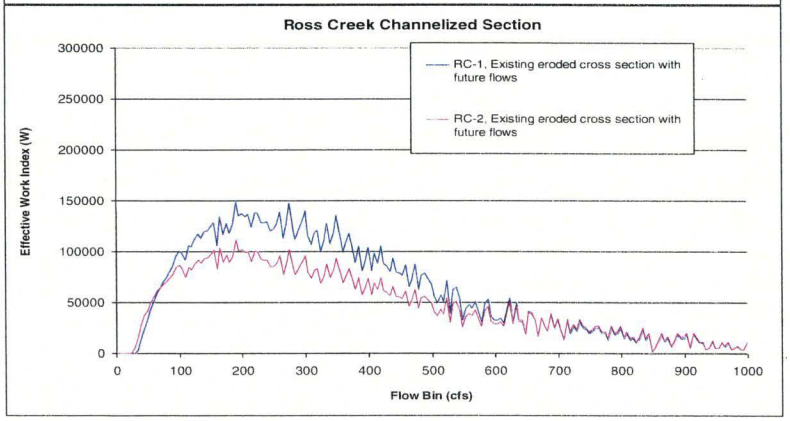
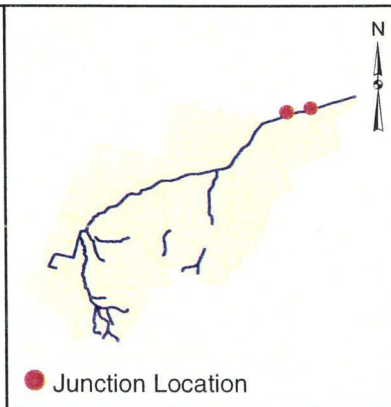


Figure 5-4
Effective Work Curves for Pre-urban, Existing, and Future Watershed Conditions for RC-1 and RC-2: Channelized Sections



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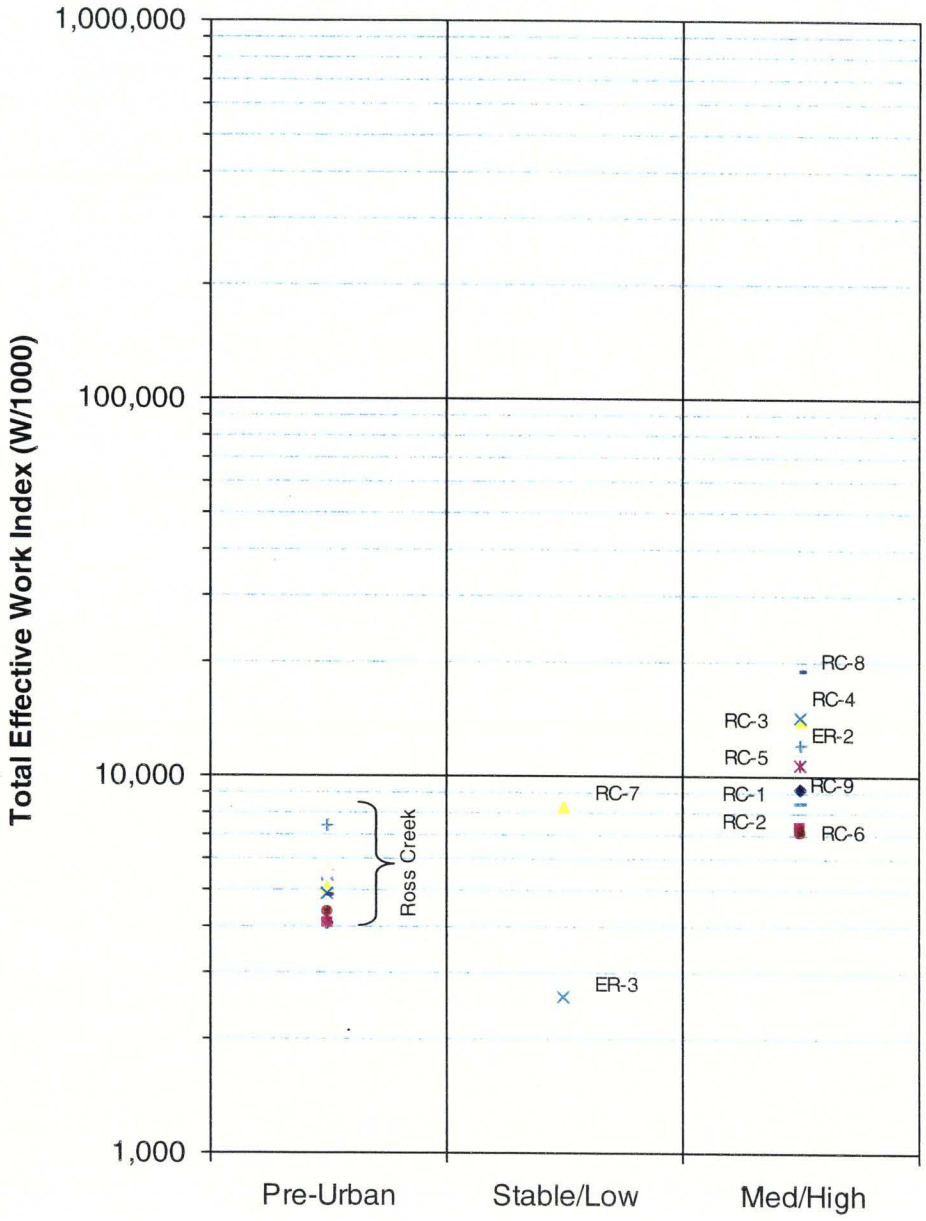


Figure 5-5
Total Effective Work Index for Ross Creek: Continuous Modeling Using Existing and Pre-Urban Flows

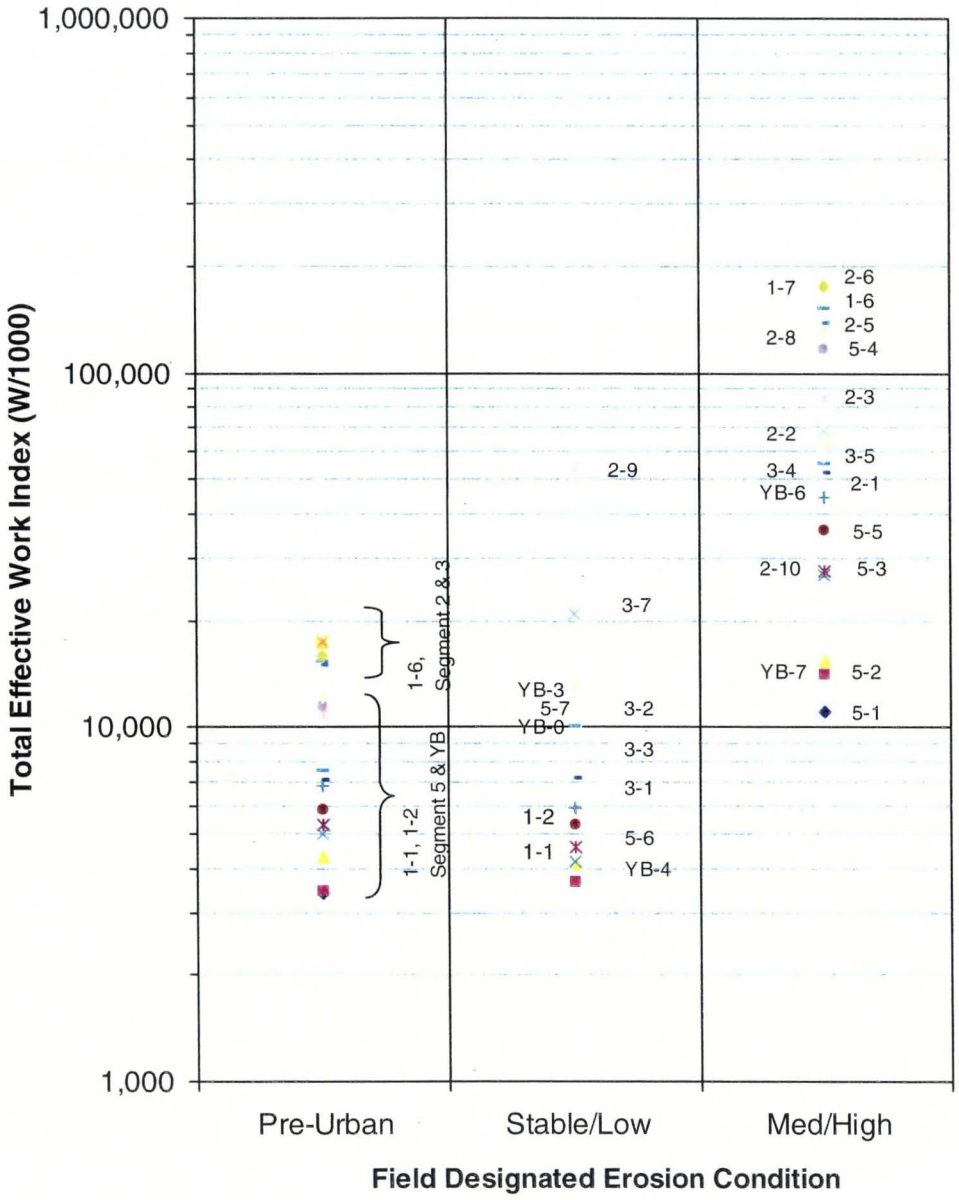


Figure 5-6
Total Effective Work Index for Thompson Creek and Yerba Buena Creek: Continuous Modeling Using Existing and Pre-Urban Flows

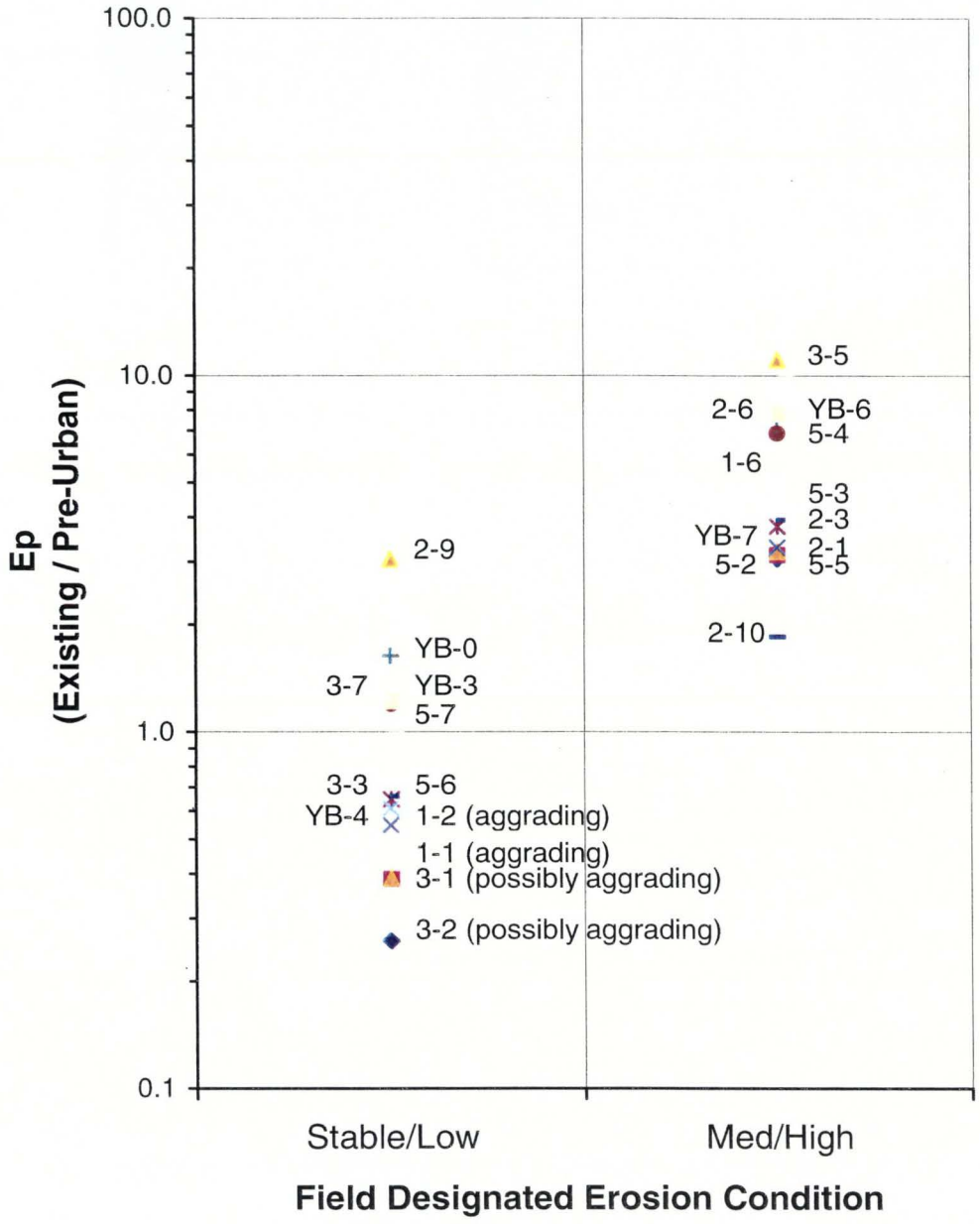


Figure 5-7
Erosion Potential Chart for Thompson Creek:
Continuous Modeling Using Existing Flows

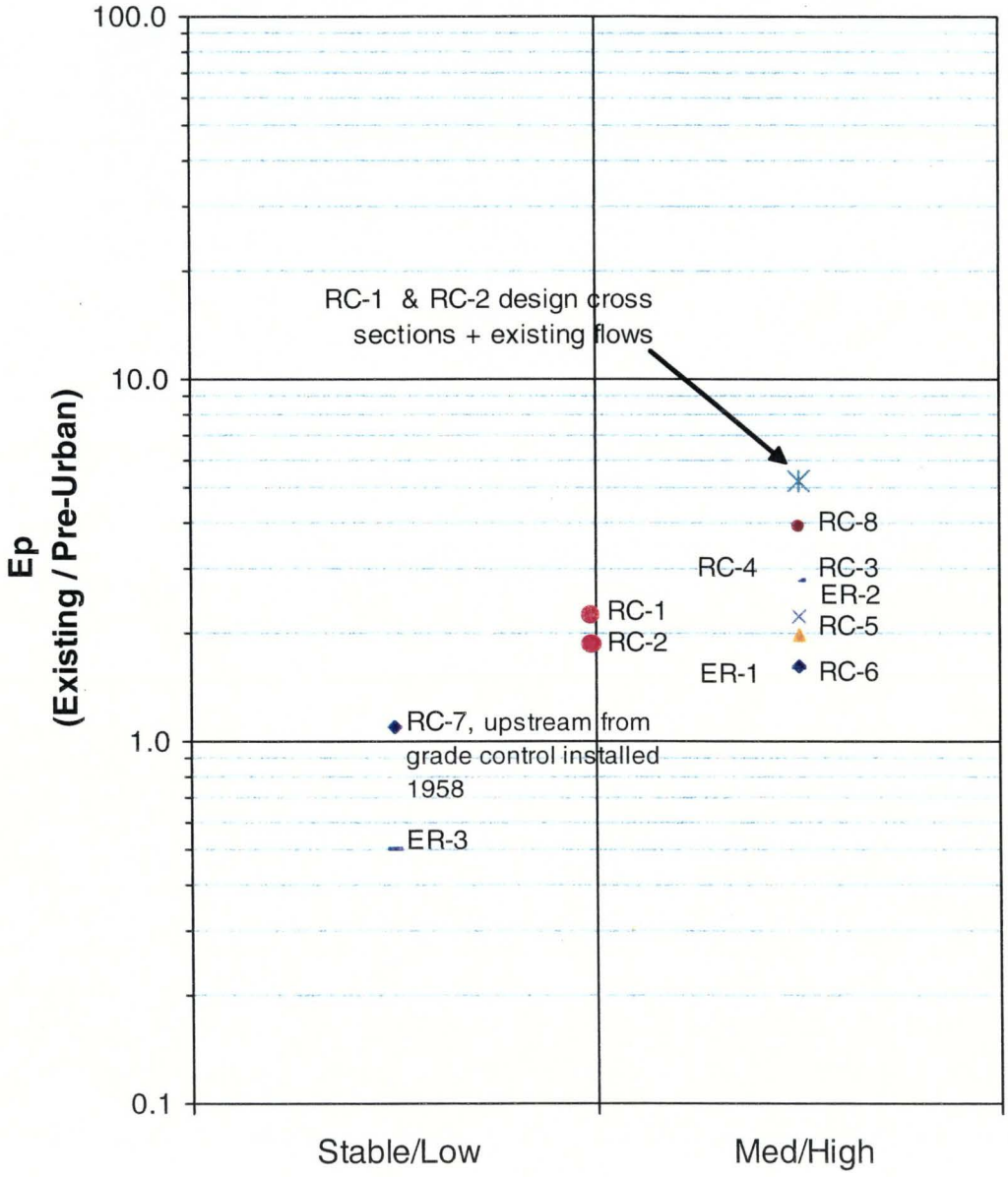


Figure 5-8
Erosion Potential Chart for Ross Creek:
Continuous Modeling Using Existing Flows

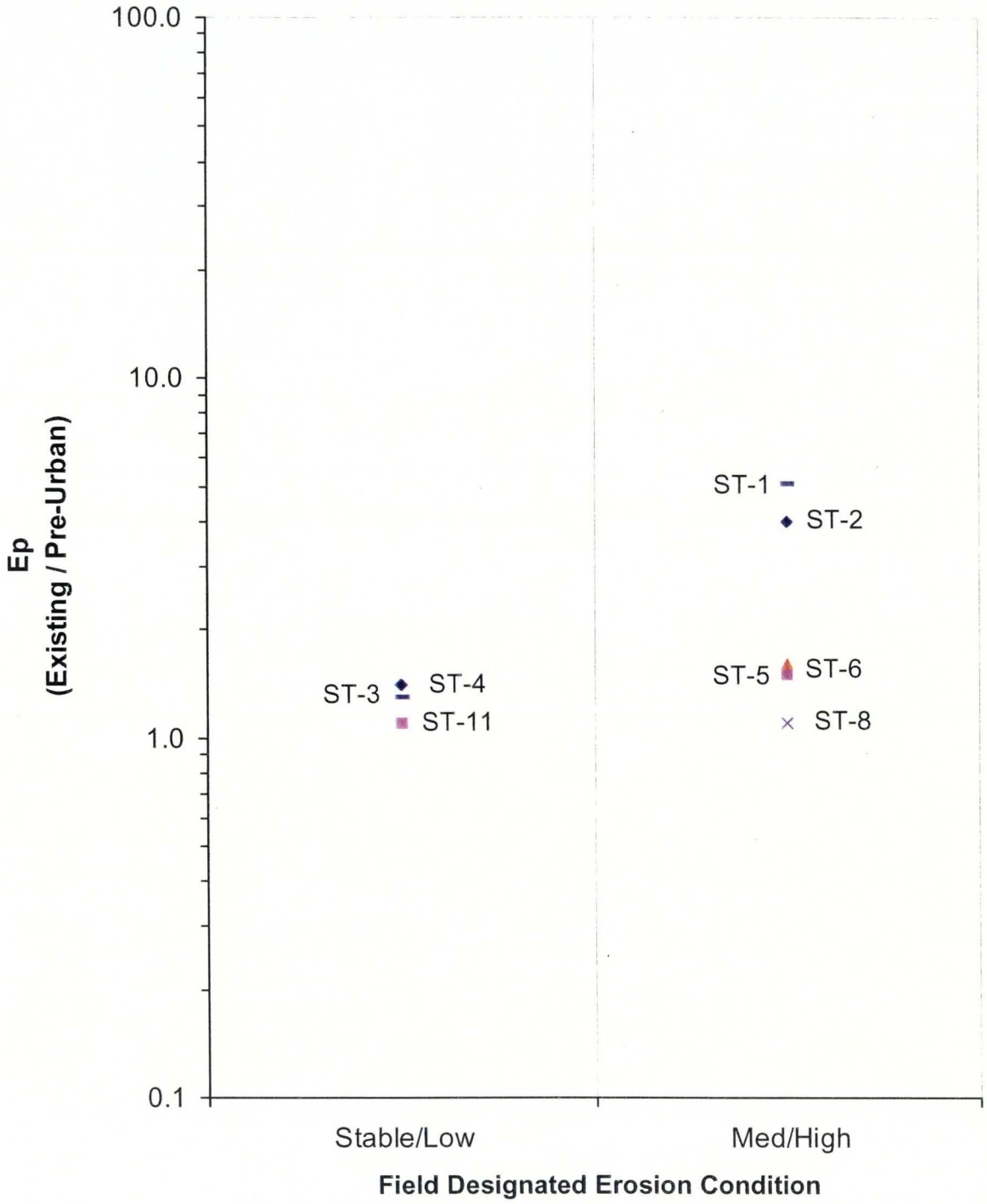


Figure 5-8s
Erosion Potential for San Tomas Creek:
Continuous Modeling Using Existing Flows

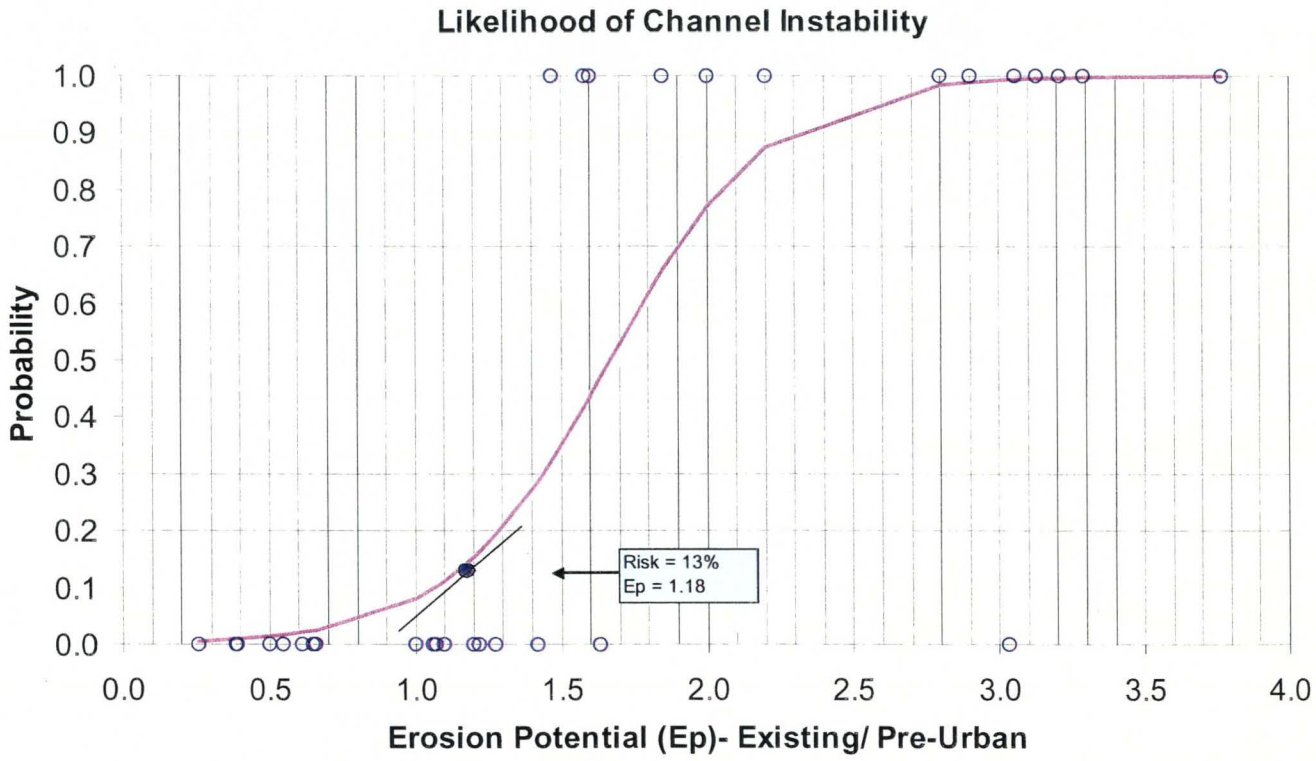
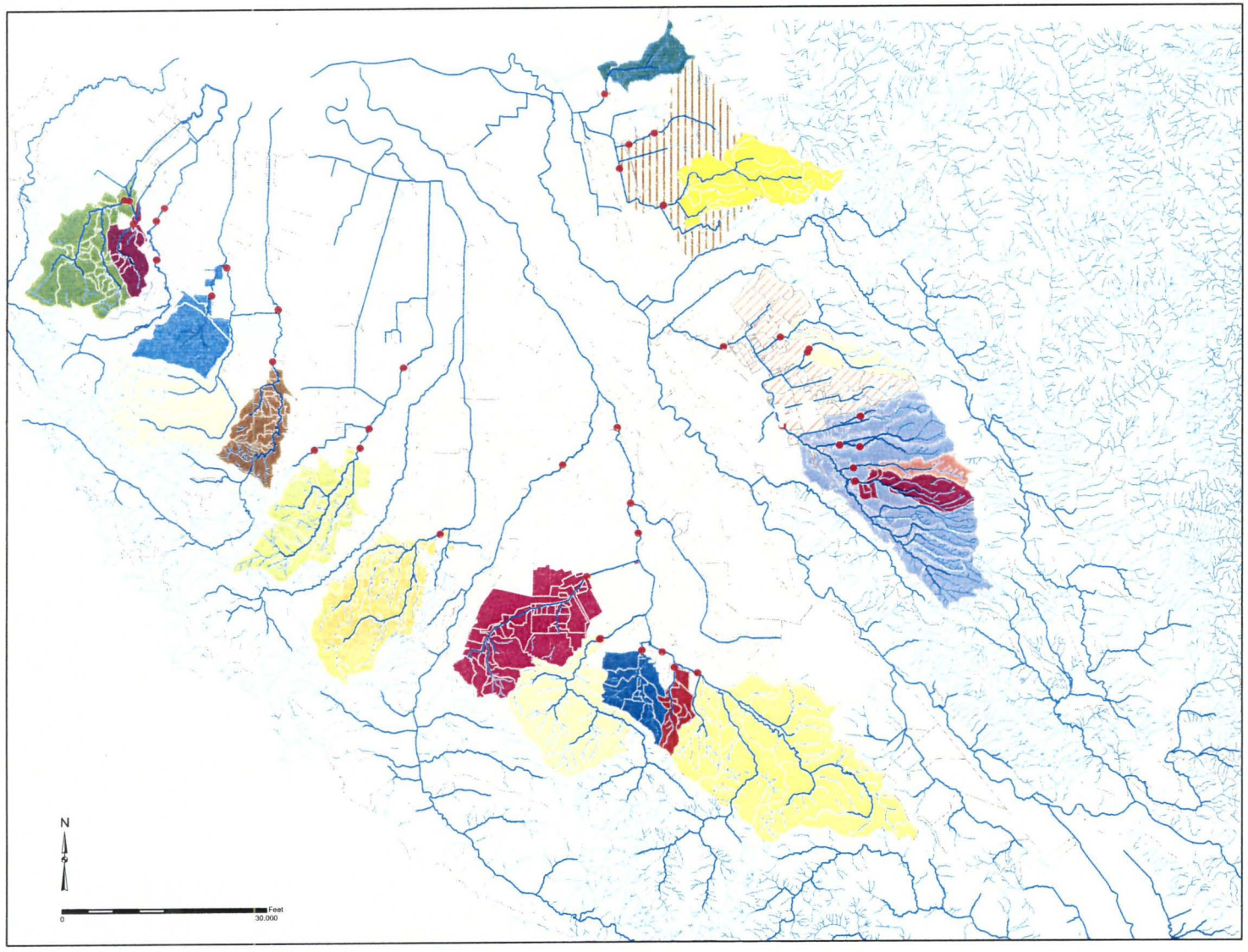


Figure 5-9
Probability Curve of the Erosion Potential for the Pooled Data of Ross Creek, San Tomas Creek, and Thompson Creek



Legend

- GPS Point
- Major Creek
- Minor Creek
- HMP Catchment Boundary
- Adobe Creek Catchment
- Alamitos Creek Catchment
- Baron Creek Catchment
- Berryessa Creek Catchment
- Calabazas Creek Catchment
- Calera Creek Catchment
- Evergreen Creek Catchment
- Flint Creek Catchment
- Golf Creek Catchment
- Greystone Creek Catchment
- Guadalupe Creek Catchment
- Hale Creek Catchment
- Lower Berryessa Creek Catchment
- Lower Silver Creek Catchment
- Matadero Creek Catchment
- Permanente Creek Catchment
- Regnart Creek Catchment
- Ross Creek Catchment
- San Tomas Creek Catchment
- South Babb Creek Catchment
- Stevens Creek Catchment
- Thompson Creek Catchment
- Yerba Buena Creek Catchment

Figure 7-1
Watershed Areas where Directly Connected Impervious Area will cause Additional Stream Segments to Destabilize

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Tables

TABLES

Table 3-1: Observed Geomorphic Reaches in the Thompson Creek Subwatershed

Code	Geomorphic Reach	Hydrographic Segment	Affected Cross-Sections[1]	Bed	Banks[2]	Remarks
A	Exposed Clay Hardpan w/ Localized Zones of Deposition	Lower Segment TC1	TC1-1 to TC1-5	Historically Incised	Localized Active Erosion	Several in-stream structures control the pattern of erosion and deposition in the reach. Incision has occurred down to the clay hardpan and has been effectively halted.
B	Historically Incised w/ Severe Bank Erosion	Upper Segment TC1 - Lower Segment TC2	TC1-6 to TC2-3	Historically Incised	Actively Eroding	Steep, vertical banks greater than 10 feet high characterize this reach that has experienced significant incision, bank erosion is severe.
C	Historically Incised w/ Bank Erosion at Bends, Sinuous Planform	Upper Segment TC2 - Lower Segment TC4	TC2-5 to TC3-7	Historically Incised	Actively Eroding	Similar to Geomorphic Reach B, but slightly more stable due to increased density of bank vegetation. Also, increased sinuosity of channel planform with excessive erosion focused at bends.
D	Channel Form Controlled by Dense Vine Vegetation	Upper Segment TC4 - Lower Segment TC5	None	Not Currently Incising	No Excessive Erosion	Extremely dense vine (blackberry) and shrub vegetation controls the stability of the reach where high flows are routed and dispersed over the dense vegetation rather than through the channel, limiting bank erosion.
E	Actively Incising w/ Bank Erosion	Mid Segment TC5	TC5-1 to TC5-5	Actively Incising	Actively Eroding	Located just downstream from some the most recent development, the channel bed in this reach is showing signs of past and present incision (exposed root mats, small knickpoints), which decreases in the upstream direction.
F	Upper End of Development, Meta-Stable, Less Disturbed	Upper Segment TC5	TC5-6 to TC5-7	Not Currently Incising	No Excessive Erosion	Located upstream from most development, this reach has relatively stable channel bed and banks, but could be impacted by a) future development and/or b) downstream incision.
G	The Gorge Reach:Canyon-like Channel w/ Landslides	Up to the headwaters "bowl"	None	Not Currently Incising	No Excessive Erosion	Periodic natural landslides probably recur, likely causing pulses of sedimentation downstream.
	Tributaries					
H	Actively Incising w/ Bank Erosion and Large Knickpoints	Segment YB1, Yerba Buena Creek	YB1-1 to YB1-7	Actively Incising	Actively Eroding	At least three large (> 5 feet) knickpoints are present along the length of Yerba Buena Creek, with some sections exhibiting very steep, canyon-like channels. Yerba Buena Creek above Villa Vista is currently stable (not incised).
I	Stable Rock- or Clay-cut Tributary Channels	Upper Evergreen Creek, Upper Yerba Buena	None	Not Currently Incising	No Excessive Erosion	Small channels cut into bedrock or hard clay with thin, immature floodplains characterize the upper sections of these tributaries, which have the potential to be impacted by downstream incision or nearby development.
J	Engineered Channels or Storm Drains	Norwood, Quimby, Fowler Creeks	None	Not Currently Incising	No Excessive Erosion	Man-made channels or pipes that convey flows to Thompson Creek that are presently stable but could be affected by incision on the main stem.

Notes:

[1] Cross-sections represent one site within a geomorphic reach and may not exhibit all characteristics that are described for the reach.

[2] Some amount of bank erosion is expected in un-disturbed stream systems, therefore, it is noted in reaches that are not experiencing severe erosion that "no excessive erosion" is occurring to distinguish natural erosion processes from accelerated erosion due to disturbance.

Table 3-2: Hydraulic and Geomorphic Characteristics of Reaches in the Thompson Creek Subwatershed

Cross-section	Top of Bank Width	Average Depth from TOB[1]	W/D Ratio[2]	Channel Slope	D^{50} [3]	Exposed Clay Hardpan on Bed?	Depth of Mobile Material	Weakest Bank Material (left bank)[4]	Weakest Bank Material (right bank)[4]	Erosion Ranking (H,M,L)[5]
	(feet)	*(feet)*	*(unitless)*	*(feet/feet)*	*(mm)*	*(Y/N)*	*(in)*			
Segment TC1										
TC1-1	16.8	4.3	3.9	0.0016	5.8	N	24	sandy gravel	silty loam and gravel	L
TC1-2	20.3	4.7	4.3	0.0040	-	Y	3-4	silt loam	sand and small gravel	L
TC1-3	23.5	10.9	2.2	0.0084	0.0082	Y	0	pebbly clay	pebbly clay	M
TC1-4	22.8	6.1	3.7	0.0170	-	Y	6	pebbly clay	pebbly clay	M
TC1-5	25.6	9.5	2.7	0.0170	-	Y	0-24	sandy clay	silty layer	H
TC1-6	52.0	11.5	4.5	0.0102	4.5	N	12-24	sandy loam with gravel	gravelly sand	M
TC1-7	11.1	3.4	3.2	0.0091	-	N	12	silt loam with gravel/sand	silty loam with gravel/sand	M
Segment TC2										
TC2-1	46.2	13.5	3.4	0.0078	-	N	18	loamy clay	loamy clay	M-H
TC2-2	36.8	12.5	3.0	0.0160	-	N	10	clayey sand-gravel	clayey sand-gravel	L-M
TC2-3	45.4	13.2	3.5	0.0109	-	N	12	clayey sand-gravel	clayey sand-gravel	H
TC2-5	15.2	3.9	3.9	0.0117	4.0	N	> 24	silt, sand and some clay	sandy silt	M
TC2-6	30.0	7.4	4.0	0.0150	-	N	10	well sorted med sand	clayey sand and gravel	M
TC2-8	34.8	10.0	3.5	0.0013	-	N	6	sand and gravel	fine sand	H
TC2-9	15.8	4.4	3.6	-	-	N	7	colluvium	fine sand w/ some clay	L
TC2-10	59.4	13.2	4.5	0.0064	3.2	N	-	sand-gravel-silt w/ clay	sand-gravel w/ some clay	M
Segment TC3										
TC3-1	17.5	4.1	4.3	0.0032	-	N	24	sand	silty loam	L-M
TC3-2	23.7	4.2	5.6	0.0031	-	N	24	silty-sandy loam	silty loam w/ pebbles	L
TC3-3	22.9	6.3	3.7	0.0045	5.0	N	24	silty loam w/ gravel	silty fine gravelly loam	L-M
TC3-4	18.7	7.0	2.7	0.0084	-	N	24	clayey pebbly with silt loam	sandy gravelly loam	M
TC3-5	22.4	4.2	5.3	0.0099	-	N	-	silty sandy loam	gravelly silty loam	H
TC3-6	10.3	3.2	3.3	0.0086	0.0065	Y	24	sandy gravelly loam	gravelly loam	H
TC3-7	13.6	3.6	3.8	0.0094	6.9	N	18	silty loam with some gravels	silty loam with some gravels	L
Segment TC5										
TC5-1	46.1	11.9	3.9	0.0250	-	N	9.6	sand and gravel with silt	silty sand w/ some gravel	H
TC5-2	13.8	5.2	2.7	0.0058	9.0	N	>24	silty sand and gravel	silty sand and gravel	M
TC5-3	11.0	7.3	1.5	0.0047	-	N	12	silts and sands	fine sand and silt	H
TC5-4	13.9	4.4	3.2	0.0166	-	N	>24	clayey sand and gravel	silty sand	H
TC5-5	11.7	2.3	5.2	0.0102	3.7	N	>>24	pebbly sand and silt	sand, pebbly sand	M-H
TC5-6	19.0	3.6	5.2	0.0086	8.0	N	12	well sorted sand	sand and gravel	S
TC5-7	14.0	1.8	7.8	0.0129	3.5	N	>24	clayey sand and gravel	clayey sand and gravel	S
Segment YB1										
YB1-0	4.4	0.9	4.7	0.0222	-	N	24	pebbly silty loam	pebbly silty loam	S
YB1-1	4.9	2.2	2.3	0.0179	10.2	N	12	silty pebbly loam w/ gravels	silty pebbly loam w/ gravels	M
YB1-2	7.9	2.2	3.5	0.0282	-	N	12	silty pebbly loam	silty pebbly loam	M
YB1-3	8.9	2.9	3.1	0.0221	-	N	24	silty pebbly loam w gravels	silty pebbly loam	M
YB1-4	8.2	1.6	5.1	0.0111	-	N	24	silty loam -- gravelly cobbly sand	gravelly cobbly loam	L
YB1-5	unsurveyed			-	-	Y	12	cobbly, silty, pebbly loam	cobbly, silty, pebbly loam	L
YB1-6	10.8	9.4	1.2	0.0139	0.028	Y	12	clay-silty gravelly-cobbly loam	clayey-gravelly loam, cobbly silty loam	H
YB1-7	27.7	2.5	11.0	0.0149	4.8	N	24	silty gravelly loam	silty gravelly loam	M

Notes:

[1] TOB = Top of Bank

[2] W/D ratio for full channel dimensions to TOB, not bankfull

[3] D^{50} = the median diameter of bed material determined from a samples collected from the bed surface.

[4] left/right bank = described from the observer facing downstream

[5] Erosion Ranking: S=stable, L = low, M = medium, H = high; based on bank, cross-section, and reach-wide observations/measurements

Table 3-3: Grain Size Data for Stream Bed and Bedload Material, Thompson Creek Subwatershed

Sample Location[1, 2]	Cross-sections with similar bed material [3]	D-Size[4]				Percent Gravel	Percent Sand	Percent Fines	
		D-10	*D-30*	*D-50*	*D-60*				
Unconsolidated Bed Material									
TC1-1	TC1-1	0.616	1.98	5.8	8.26	53.9	44.7	1.4	
TC1-6	TC1-5 to TC2-3	0.704	1.82	4.5	6.96	48.7	50.4	0.9	
200' u/s TC2-5	TC2-5 to TC2-8	0.581	1.56	4.0	6.62	46.5	53.2	0.3	
50' d/s TC2-10	TC2-9 to TC2-10	0.541	1.33	3.2	5.37	42.8	56.9	0.3	
TC3-3	TC3-1 to TC3-3	0.677	2.38	5.0	6.63	51.7	46.7	1.6	
TC3-7	TC3-4 to TC3-7	1.09	3.31	6.9	8.91	60.4	38.9	0.7	
TC5-2	TC5-1 to TC5-3	1.18	3.82	9.0	13	65.4	34.2	0.4	
TC5-5	TC5-4 to TC5-5	0.858	1.93	3.7	5.56	43.8	55.8	0.4	
TC5-6	TC5-6	1.17	3.98	8.0	10.8	65.6	34	0.4	
50' u/s TC5-7	TC5-7	0.236	1.12	3.5	5.8	44.2	51.6	4.2	
YB1-7	YB1-1 to YB1-2	0.443	1.67	4.8	7	49.8	49.2	1.0	
YB1-1	YB1-3, 1-4, 1-7, and 1-0	0.724	4.92	10.2	15.1	70.6	28.1	1.3	
Cohesive Bed Material								*% Silt*	*%Clay*
TC1-3	TC1-2 to TC1-3	--	0.0016	0.0082	0.021	2.7	23.6	41.1	32.6
TC3-6	Small exposure at base of bank	--	0.0015	0.0065	0.011	2.3	19	45.9	32.8
YB1-6	YB1-5 and YB1-6	--	0.0048	0.028	0.0448	0.1	29.7	46.4	23.8
Bedload[5]								Percent Fines	
TCQR	--	0.373	1.02	1.9	2.55	22	76.7	1.3	
YBSF	--	0.364	2.91	8.0	10.1	63.6	34.6	1.8	

[1] Samples were sieved at Cooper Testing Lab. Hydrometer analysis was performed on the cohesive bed material.

[2] Locations are listed from downstream to upstream within each category (unconsolidated, etc.). Segment YB1 enters Thompson Creek between segments TC4 and TC5.

[3] This column lists cross-section that have similar bed characteristics to the listed sample. These designations will be used to generalize bed conditions in the stability model.

[4] Size, in millimeters, for which 10, 30, 50, and 60 percent of the sample is finer.

[5] Bedload samples were collected at high flows during the storm on December 17, 2002. TCQR is 200 feet u/s of TC1-2; YBSF is near YB1-7. These samples were sieved at the lab and included to compare bedload (transport) to bed material (storage).

Table 3-4: Measurements of Depth to Existing Channel Bed and Banks from Dated Structures at Thompson Creek

Hydrographic Segment	Geomorphic Reach	Location	Type of structure	Date of structure	Depth from base of structure to current channel bed[1] *(feet)*	Distance from structure to current bank[2] *(feet)*
TC1	A	Upstream from the Quimby Road weir	Pipe		~2.0	-
TC2	B	Quimby Creek confluence; between cross-sections TC1-7 and TC2-1	Sackrete grade control structure		2.6 - 3.0	-
TC2	C	Between cross-sections TC2-6 and TC2-8	Old weir		2.3	-
TC2	C	Between cross-sections TC2-9 and TC2-10	Pipe		1.6	-
TC5	E	Downstream from The Villages subdivision; between cross-sections TC5-2 and TC5-3	Bridge	1943	3.2	-
TC5	E	San Filipe Road crossing near The Villages subdivision; between cross-sections TC5-4 and TC5-5	Outfall structure		3.1	4.1

Notes:

[1] To gain some understanding of the amount of incision in a particular portion of Thompson Creek, depths from old structures (believed to be at grade in the past) to the current channel bed were measured. These measurements do not indicate an absolute amount of incision and should be taken as estimates of the range of incision over a length of stream. It is important to note that channel bed incision and bank erosion are usually most severe around in-channel obstructions, such as bridge abutments, outfall structures and grade control structures.

[2] Where possible, the distance from an old structure that was once even with the channel bank to the current bank was measured to estimate bank retreat. Only one of these measurements was taken for the Thompson Creek subwatershed and such measurements must be used carefully because localized bank erosion generally occurs at the contact between natural bank material and hardened bank protection.

Table 3-5: Observed Geomorphic Reaches in the Ross Creek Subwatershed

Code	Geomorphic Reach	Affected Cross-Sections[1]	Bed	Banks[2]	Remarks
A	Earthen Trapezoid Channel with Meandering Low Flow Channel and Inchannel "Bench"	RC1-2	Historically Incised	No Excessive Erosion	Channelized in 1955, channel has incised through altered bed creating a very prominent, linear "bench", has a meandering low flow channel and pool/riffle sequences, semi-stable
B	Earthen Trapezoid Channel with Steep Channel Banks and Severe Bank Erosion (Widening)	RC3	Historically Incised	Acitvely Eroding	Severe bank erosion, steep bank slopes, not much evidence of the "bench" seen downstream, channel bed is much wider than Reaches A and C
C	Earthen Trapezoid Channel Punctuated with Grade Control Structures	RC4-6	Historically Incised	Acitvely Eroding	Channelized in 1957, channel has incised through altered bed creating a very prominent, linear "bench", reach is punctuated by grade control structures, localized bed and bank erosion dictated by in-stream structures
D	Non-engineered Channel with Numerous Bank Protection Projects, Low Erosion	RC7	Minor Active Incision	No Excessive Erosion	Unchannelized upper section of creek, numerous bank protection projects, trees located on lower banks, some undercutting but tree roots act as a stabilizing factor
E	Non-engineered Channel with Active Bank Widening	RC8	Minor Active Incision	Acitvely Eroding	Active bank widening, exposed tree roots, loss of vegetation, localized erosion
	East Ross Creek Tributary				
F_1	Active Incision and Bank Erosion in Lower Reach	ER1-2	Actively Incising	Acitvely Eroding	Some alteration (widening) has been conducted on the channel for flood control purposes, active incision and bank erosion, some pipes exposed up to 1.5 feet, steep banks
F_2	Stable Upper Reach of Tributary	ER3	Not Currently Incising	No Excessive Erosion	Stable channel with meandering, vegetated banks, low erosion, debris jam

Notes:

[1] Cross-sections represent one site within a geomorphic reach and may not exhibit all characteristics that are described for the reach.

[2] Some amount of bank erosion is expected in undisturbed stream systems, therefore, it is noted in reaches that are not experiencing severe erosion that "no excessive erosion" is occurring to distinguish natural erosion processes from accelerated erosion due to disturbance.

Table 3-6: Hydraulic and Geomorphic Characteristics of Reaches in the Ross Creek Subwatershed

Cross-section	Top of Bank Width	Average Depth from TOB[1]	W/D Ratio	Channel Slope	"n" [2]	D-50 [3,4]	Depth of Mobile Material	Weakest Bank Material	Primary Erosion Mechanism (Sh, U, Sl)[5]	Local or Reach-wide erosion?	Active Incision?	Active Aggradation?	Erosion Ranking (H,M,L)[6]
	(feet)	*(feet)*	*(unitless)*	*(feet/feet)*	*(unitless)*	*(mm)*	*(in)*						
Ross Creek													
RC-1	37.0	8.6	4.3	0.0039	0.032	*15*	8	fine sand/silt	Sh	Reach-wide	N	N	M/L
RC-2	39.0	9.2	4.2	0.0067	0.036	--	14	gravel in matrix of sandy silt	Sh, U	Reach-wide	N	N	M/L
RC-3	38.4	9.3	4.1	0.0061	0.039	--	16	sandy gravel	Sh, U, Sl	Reach-wide	N	N	H
RC-4	37.5	9.2	4.1	0.0060	0.030	*20*	7	slightly cohesive sand/silt/clay	Sh, U	Reach-wide	2.0'	N	M
RC-5	41.0	10.4	3.9	0.0049	0.032	--	16	cobbles in a sandy/silt matrix	Sh, U, Sl	Reach-wide	Unsure	N	M
RC-6	33.7	9.4	3.6	0.0039	0.032	--	18	cobbles in a sand/clay matrix	Sh	Local	N	N	M/L
RC-7	25.7	6.0	4.3	0.0145	0.046	**30.4**	6	sandy clay with some gravels	U (minor)	Local	N	N	L
RC-8	24.9	6.6	3.8	0.0078	0.041	*14*	>12	sand/silt with some gravels/cobbles	Sl	Reach-wide	0.5	N	H
East Ross Creek													
ERC-1	19.3	6.5	3.0	0.0098	0.033	*21*	--	cohesive sand/silt/clay	Sh, U, Sl	Reach-wide	1'-2'	N	H
ERC-2	15.1	3.6	4.1	0.0187	0.048	--	--	cohesive clays/sand/gravels	Sh, U	Reach-wide	0.5'-1.5'	N	M/H
ERC-3	4.3	0.7	6.4	0.0191	0.045	*1.9*	6	semi-cohesive clay/silt/gravels	none	--	N	0.5'-0.75' (d/s)	L

Notes:

[1] TOB = Top of Bank

[2] Channel roughness or Manning's "n" value using the method presented by Cowan (1956)

[3] D-50 = the median diameter of bed material determined from a sample/count

[4] D-50 in *italics* indicates analysis by volume (bed core), where as a D-50 in **bold** indicates analysis by area (Wolman pebble count)

[5] Primary Erosion Mechanism: Sh = bank shear, U = Undercutting, Sl = bank slumping

[6] Erosion Ranking: L = low, M = medium, H = high; based on bank, cross-section, and reach-wide observations/measurements

Table 3-7: Grain Size Data for Stream Bed and Bedload Material, Ross Creek Subwatershed

Sample Location[1]	Cross-sections with Similar Bed Material[4]	D-Size[5]				Percent > 3"	Percent Gravel	Percent Sand	Percent Fines
		D-10	*D-30*	*D-50*	*D-60*				
Ross Creek									
RC-1[2]	RC-2, RC-3	1.8	7.3	15.0	23.8	0.0	76.8	23.1	0.1
RC-4[2]	RC-5	2.5	8.9	20.0	29.1	0.0	81.5	18.4	0.1
RC-7[3]		5.0		30.4		0.0			
RC-8[2]	RC-6	2.1	7.0	15.0	23.3	0.0	78.0	21.6	0.4
East Ross Creek									
ERC-1[2]	ERC-2	1.8	10.9	23.0	29.0	0.0	82.3	17.1	0.6
ERC-3[2]		0.2	0.7	1.7	3.3	0.0	33.0	62.3	4.7

Notes:

[1] Locations are listed from downstream to upstream within each sub-watershed

[2] Samples were sieved at Cooper Testing Lab

[3] Analyzed by the pebble count method

[4] This column lists cross-sections that have similar bed characteristics to the listed sample. These designations will be used to generalize bed conditions in the model.

[5] Size, in millimeters, for which 10, 30, 50, and 60 percent of the sample is finer.

Table 3-8: Hydraulic and Geomorphic Characteristics of Reaches in the San Tomas Creek Subwatershed

Cross-section	Top of Bank Width	Average Depth from TOB[1]	W/D Ratio	Channel Slope	"n" [2]	D-50 [3,4]	Depth of Mobile Material	Weakest Bank Material	Primary Erosion Mechanism (Sh, U, Sl)[5]	Local or Reach-wide erosion?	Active Incision?	Active Aggradation?	Erosion Ranking (H,M,L)[6]
	(feet)	*(feet)*	*(unitless)*	*(feet/feet)*	*(unitless)*	*(mm)*	*(in)*						
San Tomas Creek													
STC-1	31.9	9.8	3.2	0.0008	0.030	*12*	>18	sandy clay w/ cobbles/gravels	Sh, U	Reach-wide	2'	N	H
STC-2	37.8	11.4	3.3	0.0036	0.039	--	>12	gravel in a sandy/silty/clay matrix	Sh, U	Reach-wide	N	N	H
STC-3	21.4	2.9	7.3	0.0066	0.039	*13*	--	unconsolidated sand/silt/soil	U	Reach-wide	N	unsure	L
STC-4	18.6	1.7	10.7	0.0153	0.039	--	12	silty material some gravels	none	--	N	N	L
STC-5	19.2	6.2	3.1	0.0012	0.042	--	17	sandy clay-gravelly layers	Sh, U	Reach-wide	>1.0'	N	H
STC-6	22.0	4.9	4.5	0.0119	0.042	*24*	>12	unconsolidated silty colluvium/soil	Sl, U, Sh	Reach-wide	N	N	M
STC-7	12.2	2.1	5.7	0.0210	0.044	--	9	sandy matrix with graves/cobbles	U, Sh	Reach-wide	0.5'	N	M
STC-8	25.4	5.1	5.0	0.0099	0.044	--	12	clay/silt loam with gravels/cobbles	U	Local	0-0.5'	local	M
STC-9	19.0	7.7	2.5	0.0196	--	**25.8**	14	fine gravels	Sl, Sh, U	Reach-wide	0.5'-2'	N	H
STC-10	14.7	4.3	3.4	0.0255	0.040	**27.2**	>12	sandy matrix with graves/cobbles	Sl, Sh	Reach-wide	N	N	M/H
STC-11	13.2	2.0	6.5	0.0327	0.042	**32.4**	2	gravelly/silty sand	none	--	N	N	L
STC-12	--	--	--	0.0856	0.045	--	--	sandy matrix with graves/cobbles	U	Local	N	N	L

Notes:

[1] TOB = Top of Bank

[2] Channel roughness or Manning's "n" value using the method presented by Cowan (1956)

[3] D-50 = the median diameter of bed material determined from a sample/count

[4] D-50 in *italics* indicates analysis by volume (bed core), where as a D-50 in **bold** indicates analysis by area (Wolman pebble count)

[5] Primary Erosion Mechanism: Sh = bank shear, U = Undercutting, Sl = bank slumping

[6] Erosion Ranking: L = low, M = medium, H = high; based on bank, cross-section, and reach-wide observations/measurements

Table 3-9: Grain Size Data for Stream Bed and Bedload Material, San Tomas Creek Subwatershed

Sample Location[1]	Cross-sections with Similar Bed Material [4]	D-Size[5]				Percent > 3"	Percent Gravel	Percent Sand	Percent Fines
		D-10	*D-30*	*D-50*	*D-60*				
San Tomas Creek									
STC-1[2]		2.6	6.1	13.0	20.2	0.0	76.7	23.3	0.0
STC-3[2]		0.7	3.9	14.0	25.6	0.0	68.0	31.7	0.3
STC-6[2]	ST-5	0.8	9.4	22.0	33.6	9.8	69.1	20.4	0.7
STC-9[3]		1.3		25.8					
STC-10[3]		3.8		27.2					
STC-11[3]		10.7		32.4					

Notes:

[1] Locations are listed from downstream to upstream within each sub-watershed.

[2] Samples were sieved at Cooper Testing Lab

[3] Anylized by the pebble count method

[4] This column lists cross-sections that have similar bed characteristics to the listed sample. These designations will be used to generalize bed conditions in the model.

[5] Size, in millimeters, for which 10, 30, 50, and 60 percent of the sample is finer.

Table 4-1: Drainage Area Parameterization for SMA Modeling in HEC-HMS in the Lower Silver-Thompson Creek Subwatershed

		Area (mi^2)			Canopy Interception (in)			Surface Depression (in)			Soil Infiltration (in/hr)			Soil Profile Capacity (in)			Tension Zone Capacity (in)			Percolation (in/hr)		
Subbasin ID	Type	Existing	Future	Pre-Developed	Existing	Future	Pre-Developed	Existing	Future	Pre-Developed	Existing	Future	Pre-Developed	Existing	Future	Pre-Developed	Existing	Future	Pre-Developed	Existing	Future	Pre-Developed
0	I	0.240	0.274	0.490	0.00	0.00	0.26	0.24	0.24	0.38	0.00	0.00	0.13	0.00	0.00	11.68	0.00	0.00	4.87	0.00	0.00	0.08
	P	0.250	0.215		0.23	0.23		0.38	0.38		0.11	0.11		12.28	12.28		5.14	5.14		0.07	0.07	
1	I	0.124	0.203	0.391	0.00	0.00	0.26	0.28	0.28	0.38	0.00	0.00	0.20	0.00	0.00	12.67	0.00	0.00	5.73	0.00	0.00	0.14
	P	0.267	0.188		0.24	0.24		0.38	0.38		0.21	0.21		13.11	13.11		5.95	5.95		0.14	0.14	
2		1.260	1.260	1.260	0.24	0.19	0.24	0.38	0.38	0.38	0.25	0.20	0.25	8.31	6.80	8.31	3.23	2.56	3.23	0.17	0.13	0.17
3		1.238	1.238	1.238	0.25	0.22	0.25	0.38	0.37	0.38	0.22	0.19	0.22	8.22	7.45	8.22	3.29	2.94	3.29	0.15	0.13	0.15
4		1.069	1.069	1.069	0.24	0.22	0.24	0.37	0.37	0.37	0.16	0.15	0.16	9.29	8.73	9.29	3.79	3.53	3.79	0.11	0.10	0.11
5	I	1.524	1.724	2.195	0.00	0.00	0.16	0.32	0.32	0.38	0.00	0.00	0.25	0.00	0.00	14.87	0.00	0.00	6.86	0.00	0.00	0.17
	P	0.671	0.471		0.21	0.21		0.39	0.39		0.25	0.25		15.01	15.01		6.90	6.90		0.17	0.17	
6		0.154	0.154	--	0.22	0.19	--	0.52	0.50	--	0.07	0.06		6.90	6.17	--	2.86	2.50	--	0.04	0.04	--
7		0.266	0.266	0.266	0.35	0.34	0.15	0.85	0.85	0.38	0.14	0.13	0.17	12.03	11.84	15.03	5.45	5.36	6.94	0.09	0.09	0.11
8		0.882	0.882	--	0.21	0.20	--	0.41	0.41	--	0.06	0.06	--	11.92	11.53	--	4.69	4.71	--	0.04	0.04	--
9		0.030	0.030	0.030	0.23	0.17	0.15	0.37	0.37	0.38	0.11	0.08	0.12	13.97	10.78	15.14	6.45	4.85	7.02	0.07	0.05	0.08
10		1.064	1.064	1.064	0.25	0.23	0.25	0.37	0.37	0.37	0.14	0.12	0.14	10.78	9.88	10.78	4.45	4.04	4.45	0.09	0.08	0.09
11		1.429	1.429	1.429	0.24	0.23	0.26	0.37	0.37	0.38	0.11	0.10	0.12	11.65	10.79	12.10	4.84	4.45	5.04	0.07	0.07	0.08
12		1.215	1.215	1.215	0.20	0.19	0.24	0.36	0.36	0.37	0.13	0.12	0.15	8.53	8.23	9.88	3.43	3.29	4.04	0.09	0.08	0.10
13		0.253	0.253	--	0.12	0.08	--	0.37	0.37	--	0.21	0.14	--	9.29	6.55	--	3.74	2.47	--	0.14	0.09	--
14		0.222	0.222	--	0.23	0.23	--	0.37	0.37	--	0.07	0.06	--	12.46	11.99	--	5.22	5.01	--	0.04	0.04	--
15		0.840	0.840	--	0.16	0.15	--	0.35	0.35	--	0.05	0.05	--	9.59	9.06	--	3.90	3.64	--	0.03	0.03	--
16	I	0.547	0.594	0.808	0.00	0.00	0.12	0.31	0.31	0.36	0.00	0.00	0.10	0.00	0.00	16.75	0.00	0.00	6.76	0.00	0.00	0.07
	P	0.261	0.215		0.19	0.19		0.41	0.41		0.11	0.11		19.75	19.75		8.04	8.04		0.07	0.07	
17	I	1.850	1.963	2.436	0.00	0.00	0.09	0.32	0.32	0.34	0.00	0.00	0.25	0.00	0.00	13.51	0.00	0.00	5.66	0.00	0.00	0.16
	P	0.586	0.473		0.19	0.19		0.40	0.40		0.35	0.35		19.55	19.55		8.40	8.40		0.23	0.23	
18	I	3.069	3.196	3.973	0.00	0.00	0.10	0.32	0.32	0.34	0.00	0.00	0.18	0.00	0.00	12.54	0.00	0.00	5.43	0.00	0.00	0.12
	P	0.903	0.777		0.19	0.19		0.39	0.39		0.26	0.26		17.25	17.25		7.65	7.65		0.17	0.17	
19	I	0.248	0.248	0.305	0.00	0.00	0.15	0.30	0.30	0.38	0.00	0.00	0.09	0.00	0.00	15.20	0.00	0.00	7.07	0.00	0.00	0.08
	P	0.057	0.057		0.23	0.23		0.39	0.39		0.09	0.09		15.20	15.20		7.08	7.08		0.06	0.06	
20		0.295	0.295	--	0.10	0.08	--	0.39	0.39	--	0.15	0.11	--	5.87	4.69	--	2.22	1.64	--	0.10	0.07	--
21		1.747	1.747	1.747	0.24	0.21	0.24	0.38	0.37	0.38	0.23	0.20	0.23	7.55	6.69	7.55	2.98	2.59	2.98	0.16	0.14	0.16
22		3.352	3.352	3.352	0.24	0.22	0.24	0.38	0.38	0.38	0.14	0.13	0.14	10.50	10.00	10.50	4.32	4.10	4.32	0.09	0.09	0.09
23	I	0.948	1.054	1.438	0.00	0.00	0.17	0.30	0.30	0.38	0.00	0.00	0.35	0.00	0.00	15.39	0.00	0.00	6.09	0.00	0.00	0.23
	P	0.489	0.384		0.21	0.21		0.40	0.40		0.36	0.36		15.80	15.80		6.26	6.26		0.24	0.24	
24		0.258	0.258	--	0.25	0.18	--	0.38	0.38	--	0.19	0.14	--	13.94	10.29	--	5.56	3.98	--	0.12	0.09	--
25	II	0.063	0.079	0.114	0.00	0.00	0.19	0.32	0.32	0.38	0.00	0.00	0.20	0.00	0.00	12.22	0.00	0.00	5.51	0.00	0.00	0.13
	P	0.051	0.035		0.18	0.18		0.38	0.38		0.20	0.20		12.32	12.32		5.57	5.57		0.13	0.13	
26		1.046	1.046	1.046	0.25	0.18	0.25	0.38	0.36	0.38	0.19	0.14	0.19	10.13	7.55	10.13	4.09	2.94	4.09	0.13	0.09	0.13
27		1.843	1.843	1.843	0.25	0.23	0.25	0.38	0.38	0.38	0.25	0.23	0.25	6.92	6.37	6.92	2.70	2.44	2.70	0.17	0.15	0.17
28	I	0.029	0.044	0.060	0.00	0.00	0.17	0.24	0.24	0.38	0.00	0.00	0.07	0.00	0.00	13.51	0.00	0.00	5.70	0.00	0.00	0.05
	P	0.031	0.016		0.25	0.25		0.50	0.50		0.07	0.07		13.51	13.51		5.70	5.70		0.05	0.05	
29		0.789	0.789	0.789	0.25	0.22	0.25	0.38	0.37	0.38	0.25	0.23	0.25	6.92	6.32	6.92	2.70	2.42	2.70	0.17	0.15	0.17
30	I	0.015	0.030	0.040	0.00	0.00	0.18	0.24	0.24	0.38	0.00	0.00	0.08	0.00	0.00	13.55	0.00	0.00	5.73	0.00	0.00	0.05
	P	0.026	0.010		0.23	0.23		0.40	0.40		0.08	0.08		13.55	13.55		5.73	5.73		0.05	0.05	
31	I	0.041	0.064	0.248	0.00	0.00	0.22	0.24	0.24	0.38	0.00	0.00	0.20	0.00	0.00	8.79	0.00	0.00	3.55	0.00	0.00	0.14
	P	0.054	0.032		0.20	0.20		0.48	0.48		0.11	0.11		12.11	12.11		5.06	5.06		0.07	0.07	
32		0.372	0.372	0.372	0.24	0.22	0.24	0.39	0.39	0.39	0.25	0.22	0.25	6.80	6.24	6.80	2.64	2.39	2.64	0.17	0.15	0.17
33		0.597	0.597	0.597	0.25	0.23	0.25	0.38	0.37	0.38	0.25	0.23	0.25	6.92	6.32	6.92	2.69	2.42	2.69	0.17	0.15	0.17
34		0.717	0.717	0.717	0.25	0.22	0.25	0.38	0.36	0.38	0.25	0.22	0.25	6.88	6.25	6.88	2.69	2.39	2.68	0.17	0.15	0.17
35	I	0.025	0.065	0.108	0.00	0.00	0.16	0.26	0.26	0.38	0.00	0.00	0.24	0.00	0.00	7.17	0.00	0.00	2.81	0.00	0.00	0.17
	P	0.083	0.043		0.25	0.25		0.38	0.38		0.25	0.25		7.11	7.11		2.78	2.78		0.17	0.17	
36	I	0.039	0.039	0.072	0.00	0.00	0.15	0.27	0.27	0.38	0.00	0.00	0.25	0.00	0.00	6.93	0.00	0.00	2.70	0.00	0.00	0.17
	P	0.032	0.032		0.22	0.22		0.45	0.45		0.25	0.25		6.93	6.93		2.70	2.70		0.17	0.17	
37		0.153	0.153	--	0.08	0.06	--	0.34	0.35	--	0.12	0.10	--	3.77	3.43	--	1.26	1.07	--	0.08	0.07	--
38		0.267	0.267	0.153	0.23	0.20	0.23	0.37	0.36	0.37	0.23	0.20	0.23	6.49	5.80	6.49	2.50	2.18	2.50	0.16	0.14	0.16
39	I	0.136	0.185	0.267	0.00	0.00	0.15	0.29	0.29	0.38	0.00	0.00	0.15	0.00	0.00	13.58	0.00	0.00	5.77	0.00	0.00	0.10
	P	0.152	0.102		0.17	0.17		0.41	0.41		0.13	0.13		13.07	13.07		5.54	5.54		0.09	0.09	
40		0.232	0.232	0.287	0.16	0.09	0.15	0.39	0.41	0.38	0.39	0.23	0.54	12.24	7.76	17.07	4.19	2.47	6.13	0.26	0.15	0.36
41	I	0.271	0.312	0.831	0.00	0.00	0.24	0.32	0.32	0.38	0.00	0.00	0.28	0.00	0.00	12.66	0.00	0.00	4.95	0.00	0.00	0.19
	P	0.265	0.224		0.22	0.22		0.39	0.39		0.35	0.35		15.63	15.63		5.96	5.96		0.23	0.23	
42	I	0.989	1.176	1.878	0.00	0.00	0.22	0.30	0.30	0.38	0.00	0.00	0.15	0.00	0.00	13.16	0.00	0.00	5.59	0.00	0.00	0.10
	P	0.630	0.444		0.24	0.24		0.39	0.39		0.14	0.14		11.71	11.71		5.03	5.03		0.09	0.09	
43		0.047	0.047	--	0.25	0.23	--	0.37	0.37	--	0.07	0.07	--	13.02	12.01	--	5.47	5.01	--	0.05	0.04	--
44	I	1.060	1.060	1.562	0.00	0.00	0.17	0.30	0.30	0.38	0.00	0.00	0.09	0.00	0.00	13.76	0.00	0.00	5.71	0.00	0.00	0.06
	P	0.301	0.301		0.20	0.20		0.41	0.41		0.09	0.09		14.35	14.35		5.91	5.91		0.06	0.06	
45	I	0.735	0.794	2.043	0.00	0.00	0.20	0.30	0.30	0.37	0.00	0.00	0.09	0.00	0.00	18.44	0.00	0.00	7.46	0.00	0.00	0.06
	P	0.427	0.368		0.18	0.18		0.39	0.39		0.11	0.11		21.50	21.50		8.63	8.63		0.07	0.07	
46	I	0.312	0.325	0.727	0.00	0.00	0.20	0.30	0.30	0.35	0.00	0.00	0.07	0.00	0.00	12.64	0.00	0.00	5.16	0.00	0.00	0.04
	P	0.193	0.180		0.22	0.22		0.39	0.38		0.08	0.08		15.53	15.53		6.43	6.43		0.05	0.05	
47	I	1.105	1.143	2.385	0.00	0.00	0.16	0.30	0.30	0.34	0.00	0.00	0.05	0.00	0.00	12.06	0.00	0.00	4.76	0.00	0.00	0.03
	P	0.435	0.401		0.20	0.20		0.38	0.38		0.08	0.08		19.33	19.33		7.76	7.76		0.05	0.05	
48	I	1.494	1.622	2.307	0.00	0.00	0.13	0.31	0.31	0.36	0.00	0.00	0.49	0.00	0.00	19.59	0.00	0.00	8.28	0.00	0.00	0.32
	P	0.560	0.432		0.21	0.21		0.41	0.41		0.58	0.58		23.19	23.19		9.88	9.88		0.38	0.38	
49		0.234	0.234	0.232	0.23	0.19	0.26	0.37	0.36	0.39	0.21	0.18	0.24	6.83	5.97	7.43	2.65	2.25	2.93	0.14	0.12	0.16

Note:
1) I = value for impervious area, P = value for pervious area

Table 4-2: Drainage Area Parameterization for SMA Modeling in HEC-HMS in the Ross Creek Subwatershed

Subbasin ID	Type[1]	Area (mi^2)		
		Existing	Future	Pre-Urban[2]
0	I	0.150	0.150	0.223
	P	0.073	0.073	
1	I	0.432	0.432	0.657
	P	0.225	0.225	
2	I	0.656	0.656	1.042
	P	0.386	0.386	
3	I	0.570	0.570	0.897
	P	0.327	0.327	
4	I	0.683	0.683	1.205
	P	0.522	0.522	
5	I	0.622	0.622	1.060
	P	0.438	0.438	
6	I	0.184	0.184	0.394
	P	0.210	0.210	
7	E	0.546	0.546	0.546
8	I	0.220	0.220	0.494
	P	0.274	0.274	
9	I	0.107	0.107	0.195
	P	0.088	0.088	
10	E	0.150	0.150	0.150
11	I	0.097	0.097	0.173
	P	0.076	0.076	
12	R	0.810	0.810	0.810
13	I	0.148	0.148	0.331
	P	0.183	0.183	
14	R	0.600	0.600	0.600
15	R	0.191	0.191	0.191
16	R	0.473	0.473	0.473
17	R	0.180	0.180	0.180
	TOTAL:	9.620	9.620	9.620

Notes:

1) I = Impervious urban; P = Pervious urban; E = Pervious urban external of storm drain system; R = Rural

2) All pre-urban subbasins are considered rural.

Table 4-3: Drainage Area Curve Numbers and Initial Abstractions Losses

		Runoff Curve Number (CN)			*Initial Abstraction*		
Subbasin ID	**Type**	**Existing**	**Future**	**Pre-Developed**	**Existing**	**Future**	**Pre-Developed**
0	I	98.0	98.0	76.6	0.05	0.05	0.61
	P	77.8	77.8		0.57	0.57	
1	I	98.0	98.0	73.2	0.05	0.05	0.73
	P	73.3	73.3		0.73	0.73	
2		71.7	77.5	71.7	0.79	0.58	0.79
3		72.7	75.4	72.7	0.75	0.65	0.75
4		75.6	77.4	75.6	0.64	0.58	0.64
5	I	98.0	98.0	79.6	0.05	0.05	0.51
	P	73.1	73.1		0.74	0.74	
6		80.5	83.8	--	0.48	0.39	--
7		79.8	80.5	79.8	0.51	0.49	0.51
8		81.6	82.8	--	0.45	0.42	--
9		78.7	85.4	78.7	0.54	0.34	0.54
10		76.1	78.3	76.1	0.63	0.55	0.63
11		77.3	79.2	77.0	0.59	0.52	0.60
12		79.6	81.1	75.7	0.51	0.47	0.64
13		79.9	93.0	--	0.50	0.15	--
14		79.8	80.8	--	0.51	0.47	--
15		85.0	87.9	--	0.35	0.28	--
16	I	98.0	98.0	86.2	0.05	0.05	0.32
	P	79.1	79.1		0.53	0.53	
17	I	98.0	98.0	84.3	0.05	0.05	0.37
	P	70.9	70.9		0.82	0.82	
18	I	98.0	98.0	85.4	0.05	0.05	0.34
	P	73.9	73.9		0.71	0.71	
19	I	98.0	98.0	84.6	0.05	0.05	0.36
	P	78.6	78.6		0.54	0.54	
20		83.9	95.6	--	0.38	0.09	--
21		72.4	75.9	72.4	0.76	0.64	0.76
22		76.6	78.0	76.6	0.61	0.56	0.61
23	I	98.0	98.0	77.8	0.05	0.05	0.57
	P	69.4	69.4		0.88	0.88	
24		72.7	81.7	--	0.75	0.45	--
25	I	98.0	98.0	80.0	0.05	0.05	0.50
	P	76.6	76.6		0.61	0.61	
26		73.7	80.7	73.7	0.71	0.48	0.71
27		71.9	74.4	71.9	0.78	0.69	0.78
28	I	98.0	98.0	84.3	0.05	0.05	0.37
	P	79.9	79.9		0.50	0.50	
29		72.1	74.7	72.1	0.77	0.68	0.77
30	I	98.0	98.0	83.9	0.05	0.05	0.38
	P	79.2	79.2		0.52	0.52	
31	I	98.0	98.0	78.6	0.05	0.05	0.54
	P	79.8	79.8		0.51	0.51	
32		72.4	75.1	72.4	0.76	0.66	0.76
33		71.9	74.6	71.9	0.78	0.68	0.78
34		72.0	74.9	72.0	0.78	0.67	0.78
35	I	98.0	98.0	80.3	0.05	0.05	0.49
	P	71.7	71.7		0.79	0.79	
36	I	98.0	98.0	80.5	0.05	0.05	0.48
	P	73.1	73.1		0.74	0.74	
37		87.4	97.1	--	0.29	0.06	--
38		74.0	78.0	74.0	0.70	0.57	0.70
39	I	98.0	98.0	83.1	0.05	0.05	0.41
	P	79.3	79.3		0.52	0.52	
40		72.5	95.4	72.5	0.76	0.10	0.76
41	I	98.0	98.0	74.3	0.05	0.05	0.69
	P	69.3	69.3		0.88	0.88	
42	I	98.0	98.0	79.1	0.05	0.05	0.53
	P	76.8	76.8		0.60	0.60	
43		78.8	80.7	--	0.54	0.48	--
44	I	98.0	98.0	83.5	0.05	0.05	0.39
	P	79.7	79.7		0.51	0.51	
45	I	98.0	98.0	81.4	0.05	0.05	0.46
	P	79.5	79.5		0.52	0.52	
46	I	98.0	98.0	82.4	0.05	0.05	0.43
	P	79.0	79.0		0.53	0.53	
47	I	98.0	98.0	85.4	0.05	0.05	0.34
	P	80.1	80.1		0.50	0.50	
48	I	98.0	98.0	74.6	0.05	0.05	0.68
	P	62.1	62.1		1.22	1.22	
49		74.6	79.6	71.8	0.68	0.51	0.79

Table 4-4: Time of Concentration and Storage Coefficient, Thompson Creek, Pre-Urban Conditions

Subbasin	Length (ft)	Min. Elev. (ft)	Max. Elev. (ft)	Slope (ft/ft)	n	Tc (hr)	R / (Tc + R)	R
0	6600	460	800	0.0515	0.070	0.494	0.900	4.449
1	6800	320	620	0.0441	0.070	0.520	0.900	4.679
2	19200	310	2203	0.0986	0.070	0.701	0.750	2.103
3	10500	523	2449	0.1834	0.070	0.456	0.750	1.369
4	10900	480	2403	0.1764	0.070	0.469	0.750	1.406
5	11000	135	165	0.0027	0.070	1.254	0.900	11.283
7	3200	126	134	0.0025	0.070	0.716	0.900	6.446
9	2300	125	134	0.0039	0.070	0.552	0.900	4.968
10	9800	495	2418	0.1962	0.070	0.435	0.750	1.304
11	16100	259	2384	0.1320	0.070	0.603	0.750	1.808
12	10200	243	1842	0.1568	0.070	0.467	0.750	1.401
16	9400	125	620	0.0527	0.070	0.581	0.900	5.226
17	15700	80	125	0.0029	0.070	1.465	0.900	13.182
18	17200	85	130	0.0026	0.070	1.562	0.900	14.058
19	3800	135	145	0.0026	0.070	0.767	0.900	6.904
21	14700	448	2273	0.1241	0.070	0.586	0.750	1.757
22	17700	291	2269	0.1118	0.070	0.655	0.750	1.965
23	9950	220	500	0.0281	0.070	0.691	0.900	6.220
25	2400	280	460	0.0750	0.070	0.281	0.900	2.532
26	11300	487	2341	0.1641	0.070	0.485	0.750	1.454
27	12800	664	2153	0.1163	0.070	0.557	0.750	1.672
28	2500	520	600	0.0320	0.070	0.350	0.900	3.153
29	9200	845	2480	0.1777	0.070	0.432	0.750	1.296
30	1600	460	520	0.0375	0.070	0.274	0.900	2.463
31	6350	520	920	0.0630	0.070	0.463	0.900	4.167
32	8300	689	1789	0.1325	0.070	0.441	0.750	1.323
33	9400	664	2246	0.1683	0.070	0.442	0.750	1.326
34	10900	644	2491	0.1694	0.070	0.473	0.750	1.419
35	3300	560	760	0.0606	0.070	0.344	0.900	3.092
36	2550	600	780	0.0706	0.070	0.294	0.900	2.642
38	6000	604	1417	0.1355	0.070	0.377	0.750	1.130
39	5700	420	600	0.0316	0.070	0.518	0.900	4.659
40	4200	303	503	0.0476	0.070	0.407	0.900	3.665
41	11200	320	920	0.0536	0.070	0.628	0.900	5.652
42	14400	180	1200	0.0708	0.070	0.662	0.900	5.957
44	13800	135	810	0.0489	0.070	0.708	0.900	6.369
45	13700	130	1340	0.0883	0.070	0.614	0.900	5.525
46	11500	130	880	0.0652	0.070	0.607	0.900	5.464
47	9600	125	860	0.0766	0.070	0.537	0.900	4.834
48	17100	85	500	0.0243	0.070	0.923	0.900	8.306
49	4300	552	1050	0.1158	0.070	0.334	0.900	3.007

Notes:

Overland flow Manning's roughness coefficient 0.07 represents "sparse vegetation" (SCVWD, 1998).

The ratio of R / (Tc + R) lies between .5 and .9 for pervious and rural areas (SCVWD, 1998);

a ratio of 0.75 was chosen for rural hillside open space, and 0.9 for agricultural land use.

Table 4-5: Time of Concentration and Storage Coefficient, Thompson Creek, Existing Conditions, Urban Areas

					1	2	3	4						5	6				5	7					5	8	9			
sub basin i.d.	urban area (sq mi)	urban area (sq ft)	impervious %	impervious area Ai	Street area Ast	Street width Wst	Street Length Lst	Overland Impervious							Gutter					Urban Impervious Time of Concentration TCimp	urban pervious area (to s.d.)	Overland Pervious					Urban Pervious Time of Concentration TCperv	Urban: R / (TC + R)		Urban: R	
				(sq ft)	(sq ft)	(ft)	(ft)	Li (ft)	n	L	Emin	Emax	S (ft/ft)	TCI (hr)	Lcb (ft)	n	S (ft/ft)	TCcb (hr)		(hr)	(sq ft)	Lp (ft)	n	S	TCp (hr)	(hr)	Imp	Perv	Imp	Perv
0	0.490	13647501	0.490	6688161	2729500	50	54590	36.3	0.05	5900	464	822	0.061	0.035	300	0.02	0.061	0.062		0.097	6959340	63.742	0.5	0.061	0.135	0.135	0.4	0.75	0.065	0.406
1	0.391	10886769	0.318	3456747	2177354	50	43547	14.7	0.05	200	0	20	0.100	0.020	300	0.02	0.100	0.055		0.075	7430022	85.310	0.5	0.100	0.138	0.138	0.4	0.75	0.050	0.414
5	2.195	61202223	0.694	42482502	12240445	50	244809	61.8	0.05	10100	137	377	0.024	0.056	300	0.02	0.024	0.077		0.133	1.9E+07	38.233	0.5	0.024	0.133	0.133	0.4	0.75	0.089	0.398
16	0.808	22534875	0.677	15263262	4506975	50	90140	59.7	0.05	600	0	20	0.033	0.051	300	0.02	0.033	0.071		0.122	7271613	40.335	0.5	0.033	0.126	0.126	0.4	0.75	0.082	0.377
17	2.436	67902507	0.760	51588342	13580501	50	271610	70.0	0.05	15700	67	129	0.004	0.091	300	0.02	0.004	0.117		0.208	1.6E+07	30.032	0.5	0.004	0.181	0.181	0.4	0.75	0.139	0.542
18	3.973	110751723	0.773	85565556	22150345	50	443007	71.6	0.05	17100	81	250	0.010	0.074	300	0.02	0.010	0.095		0.169	2.5E+07	28.426	0.5	0.010	0.142	0.142	0.4	0.75	0.113	0.426
19	0.305	8496108	0.814	6914808	1699222	50	33984	76.7	0.05	3900	127	147	0.005	0.089	300	0.02	0.005	0.110		0.200	1581300	23.265	0.5	0.005	0.151	0.151	0.4	0.75	0.133	0.452
23	1.438	40076604	0.660	26435547	8015321	50	160306	57.5	0.05	10000	218	521	0.030	0.051	300	0.02	0.030	0.073		0.124	1.4E+07	42.547	0.5	0.030	0.132	0.132	0.4	0.75	0.083	0.395
25	0.114	3176487	0.549	1744290	635297.4	50	12706	43.6	0.05	2400	281	463	0.076	0.036	300	0.02	0.076	0.059		0.095	1432197	56.359	0.5	0.076	0.121	0.121	0.4	0.75	0.063	0.364
28	0.060	1672605	0.478	799776	334521	50	6690	34.8	0.05	2500	522	619	0.039	0.038	300	0.02	0.039	0.069		0.107	872829	65.230	0.5	0.039	0.152	0.152	0.4	0.75	0.071	0.456
30	0.040	1123074	0.361	404973	224614.8	50	4492	20.1	0.05	1600	469	547	0.049	0.028	300	0.02	0.049	0.065		0.093	718101	79.926	0.5	0.049	0.159	0.159	0.4	0.75	0.062	0.476
31	0.095	2660994	0.433	1151730	532198.8	50	10644	29.1	0.05	3000	527	631	0.035	0.036	300	0.02	0.035	0.070		0.107	1509264	70.898	0.5	0.035	0.162	0.162	0.4	0.75	0.071	0.487
35	0.108	3013380	0.235	707868	602676	50	12054	4.4	0.05	3300	543	773	0.070	0.013	300	0.02	0.070	0.060		0.072	2305512	95.636	0.5	0.070	0.159	0.159	0.4	0.75	0.048	0.476
36	0.072	2000349	0.550	1101033	400069.8	50	8001	43.8	0.05	2400	613	812	0.083	0.036	300	0.02	0.083	0.057		0.093	899316	56.197	0.5	0.083	0.119	0.119	0.4	0.75	0.062	0.356
39	0.287	8011692	0.472	3783006	1602338	50	32047	34.0	0.05	5700	406	620	0.038	0.038	300	0.02	0.038	0.069		0.107	4228686	65.977	0.5	0.038	0.154	0.154	0.4	0.75	0.072	0.462
41	0.536	14942430	0.505	7551675	2988486	50	59770	38.2	0.05	5900	318	587	0.046	0.039	300	0.02	0.046	0.066		0.105	7390755	61.827	0.5	0.046	0.143	0.143	0.4	0.75	0.070	0.428
42	1.620	45158337	0.611	27581661	9031667	50	180633	51.3	0.05	11000	181	583	0.037	0.047	300	0.02	0.037	0.070		0.116	1.8E+07	48.653	0.5	0.037	0.134	0.134	0.4	0.75	0.077	0.403
44	1.361	37948239	0.779	29550222	7589648	50	151793	72.3	0.05	12500	137	526	0.031	0.057	300	0.02	0.031	0.072		0.129	8398017	27.663	0.5	0.031	0.107	0.107	0.4	0.75	0.086	0.321
45	1.162	32392917	0.632	20481165	6478583	50	129572	54.0	0.05	8600	128	359	0.027	0.051	300	0.02	0.027	0.075		0.126	1.2E+07	45.966	0.5	0.027	0.141	0.141	0.4	0.75	0.084	0.422
46	0.505	14067450	0.618	8699733	2813490	50	56270	52.3	0.05	9100	124	477	0.039	0.046	300	0.02	0.039	0.069		0.115	5367717	47.696	0.5	0.039	0.131	0.131	0.4	0.75	0.077	0.394
47	1.544	43055568	0.718	30925926	8611114	50	172222	64.8	0.05	8400	130	650	0.062	0.046	300	0.02	0.062	0.061		0.107	1.2E+07	35.215	0.5	0.062	0.102	0.102	0.4	0.75	0.072	0.306
48	2.054	57257856	0.727	41649327	11451571	50	229031	65.9	0.05	14700	78	310	0.016	0.064	300	0.02	0.016	0.085		0.149	1.6E+07	34.075	0.5	0.016	0.138	0.138	0.4	0.75	0.099	0.415

1 Source: Nolte & Assoc 2000, Table 1 gives street area as 20% of total area for range of urban land uses
2 Source: SCVWD Hydrology Procedures, 1998. Typical average street width = 50 ft.
3 Source: SCVWD Hydrology Procedures, 1998. Lst=Ast/Wst
4 Source: SCVWD Hydrology Procedures, 1998. Li=Ai-Ast/2Lst
5 Source: SCVWD Hydrology Procedures, 1998. Kerby-Hathaway Tc equation
6 Source: Nolte & Assoc 2000, average gutter length between catch basins = 300 ft.
7 Source: SCVWD Hydrology Procedures, 1998. TCimp= TCi + TCcb
8 Source: SCVWD Hydrology Procedures, 1998. TCperv= TCimp + TCp
9 Source: SCVWD Hydrology Procedures, 1998 (p.26, range); Nolte & Assoc 2000 uses 0.4 and 0.75.

Note: All Tc and R values less than 0.1 are rounded up to 0.1 for input into HEC-HMS.

Table 4-6: Time of Concentration and Storage Coefficient, Thompson Creek, Existing Conditions, Rural and External Pervious Areas

	Subbasin	Length (ft)	Min. Elev. (ft)	Max. Elev. (ft)	Slope (ft/ft)	n		Tc (hr)	R / (Tc + R)	R
Rural	2	19200	310	2203	0.0986	0.070		0.701	0.750	2.103
Subbasins	3	10500	523	2449	0.1834	0.070		0.456	0.750	1.369
	4	10900	480	2403	0.1764	0.070		0.469	0.750	1.406
	7	3200	126	134	0.0025	0.070		0.716	0.750	2.149
	9	2300	125	134	0.0039	0.070		0.552	0.750	1.656
	10	9800	495	2418	0.1962	0.070		0.435	0.750	1.304
	11	16100	259	2384	0.1320	0.070		0.603	0.750	1.808
	12	10200	243	1842	0.1568	0.070		0.467	0.750	1.401
	21	14700	448	2273	0.1241	0.070		0.586	0.750	1.757
	22	17700	291	2269	0.1118	0.070		0.655	0.750	1.965
	26	11300	487	2341	0.1641	0.070		0.485	0.750	1.454
	27	12800	664	2153	0.1163	0.070		0.557	0.750	1.672
	29	9200	845	2480	0.1777	0.070		0.432	0.750	1.296
	32	8300	689	1789	0.1325	0.070		0.441	0.750	1.323
	33	9400	664	2246	0.1683	0.070		0.442	0.750	1.326
	34	10900	644	2491	0.1694	0.070		0.473	0.750	1.419
	38	6000	604	1417	0.1355	0.070		0.377	0.750	1.130
	40	4200	303	503	0.0476	0.070		0.407	0.750	1.222
	49	4300	552	1050	0.1158	0.070		0.334	0.750	1.002
							plus Urban Tc-Impervious			
External	6	1900	320	609	0.1521	0.070	0.129	0.343	0.750	1.028
Pervious	8	7700	246	1342	0.1423	0.070	0.126	0.545	0.750	1.634
Subbasins	13	3400	233	502	0.0791	0.070	0.149	0.476	0.750	1.428
	14	5000	279	854	0.1150	0.070	0.115	0.474	0.750	1.423
	15	3500	212	943	0.2089	0.070	0.107	0.372	0.750	1.115
	20	5200	506	920	0.0796	0.070	0.105	0.504	0.750	1.511
	24	4600	441	1182	0.1611	0.070	0.116	0.435	0.750	1.306
	37	3500	616	963	0.0991	0.070	0.107	0.421	0.750	1.264
	43	1300	520	823	0.2331	0.070	0.129	0.291	0.750	0.872

Notes:

Overland flow Manning's roughness coefficient 0.07 represents "sparse vegetation" (SCVWD, 1998).

The ratio of R / (Tc + R) lies between .5 and .9 for pervious and rural areas (SCVWD, 1998);
a ratio of 0.75 was chosen for rural hillside open space.

Table 4-7: Time of Concentration and Storage Coefficient, Thompson Creek, Future Conditions, Urban Areas

					1	2	3	4						5	6			5	7					5	8	9			
sub basin i.d.	urban area (sq mi)	urban area (sq ft)	impervious %	impervious area Ai	Street area Ast	Street width Wst	Street Length Lst	Overland Impervious							Gutter				Urban Impervious Time of Concentration TCimp	urban pervious area (to s.d.)	Overland Pervious				Urban Pervious Time of Concentration TCperv	Urban: R = (TC + R)		Urban: R	
				(sq ft)	(sq ft)	(ft)	(ft)	Li (ft)	n	L	Emin	Emax	S (ft/ft)	TCi (hr)	Lcb (ft)	n	S (ft/ft)	TCcb (hr)	(hr)	(sq ft)	Lp (ft)	n	S	TCp (hr)	(hr)	Imp	Perv	Imp	Perv
0	0.490	13647501	0.560	7648725	2729500	50	54590	45.1	0.05	5900	464	822	0.061	0.039	300	0.02	0.061	0.062	0.101	5998776	54.944	0.5	0.061	0.126	0.126	0.4	0.75	0.067	0.379
1	0.391	10886769	0.520	5656637.8	2177354	50	43547	39.9	0.05	200	0	20	0.100	0.033	300	0.02	0.100	0.055	0.088	5230131	60.051	0.5	0.100	0.117	0.117	0.4	0.75	0.058	0.351
5	2.195	61202223	0.785	48070852	12240445	50	244809	73.2	0.05	10100	137	377	0.024	0.061	300	0.02	0.024	0.077	0.138	13131371	26.820	0.5	0.024	0.112	0.112	0.4	0.75	0.092	0.337
16	0.808	22534875	0.734	16546004	4506975	50	90140	66.8	0.05	600	0	20	0.033	0.054	300	0.02	0.033	0.071	0.125	5988871	33.220	0.5	0.033	0.115	0.115	0.4	0.75	0.083	0.344
17	2.436	67902507	0.806	54711700	13580501	50	271610	75.7	0.05	15700	67	129	0.004	0.095	300	0.02	0.004	0.117	0.212	13190807	24.283	0.5	0.004	0.163	0.163	0.4	0.75	0.141	0.490
18	3.973	110751723	0.804	89089310	22150345	50	443007	75.6	0.05	17100	81	250	0.010	0.076	300	0.02	0.010	0.095	0.171	21662413	24.449	0.5	0.010	0.132	0.132	0.4	0.75	0.114	0.397
19	0.305	8496108	0.814	6914808	1699222	50	33984	76.7	0.05	3900	127	147	0.005	0.089	300	0.02	0.005	0.110	0.200	1581300	23.265	0.5	0.005	0.151	0.151	0.4	0.75	0.133	0.452
23	1.438	40076604	0.733	29383463	8015321	50	160306	66.6	0.05	10000	218	521	0.030	0.055	300	0.02	0.030	0.073	0.128	10693141	33.352	0.5	0.030	0.118	0.118	0.4	0.75	0.085	0.353
25	0.114	3176487	0.696	2211757.7	635297.4	50	12706	62.0	0.05	2400	281	463	0.076	0.043	300	0.02	0.076	0.059	0.102	964729.3	37.964	0.5	0.076	0.101	0.101	0.4	0.75	0.068	0.302
28	0.060	1672605	0.733	1225421.4	334521	50	6690	66.6	0.05	2500	522	619	0.039	0.052	300	0.02	0.039	0.069	0.121	447183.6	33.420	0.5	0.039	0.111	0.111	0.4	0.75	0.080	0.333
30	0.040	1123074	0.748	840066.15	224614.8	50	4492	68.5	0.05	1600	469	547	0.049	0.050	300	0.02	0.049	0.065	0.115	283007.8	31.499	0.5	0.049	0.102	0.102	0.4	0.75	0.077	0.307
31	0.095	2660994	0.668	1778395.8	532198.8	50	10644	58.5	0.05	3000	527	631	0.035	0.050	300	0.02	0.035	0.070	0.121	882598.2	41.460	0.5	0.035	0.126	0.126	0.4	0.75	0.080	0.378
35	0.108	3013380	0.602	1813851.5	602676	50	12054	50.2	0.05	3300	543	773	0.070	0.040	300	0.02	0.070	0.060	0.099	1199528	49.758	0.5	0.070	0.117	0.117	0.4	0.75	0.066	0.350
36	0.072	2000349	0.550	1101033	400069.8	50	8001	43.8	0.05	2400	613	812	0.083	0.036	300	0.02	0.083	0.057	0.093	899316	56.197	0.5	0.083	0.119	0.119	0.4	0.75	0.062	0.356
39	0.287	8011692	0.644	5156041.9	1602338	50	32047	55.4	0.05	5700	406	620	0.038	0.048	300	0.02	0.038	0.069	0.117	2855650	44.554	0.5	0.038	0.128	0.128	0.4	0.75	0.078	0.384
41	0.536	14942430	0.581	8685790.1	2988486	50	59770	47.7	0.05	5900	318	587	0.046	0.043	300	0.02	0.046	0.066	0.109	6256640	52.340	0.5	0.046	0.132	0.132	0.4	0.75	0.073	0.396
42	1.620	45158337	0.726	32788557	9031667	50	180633	65.8	0.05	11000	181	583	0.037	0.052	300	0.02	0.037	0.070	0.122	12369780	34.240	0.5	0.037	0.114	0.114	0.4	0.75	0.081	0.342
44	1.361	37948239	0.779	29550222	7589648	50	151793	72.3	0.05	12500	137	526	0.031	0.057	300	0.02	0.031	0.072	0.129	8398017	27.663	0.5	0.031	0.107	0.107	0.4	0.75	0.086	0.321
45	1.162	32392917	0.683	22136303	6478583	50	129572	60.4	0.05	8600	128	359	0.027	0.054	300	0.02	0.027	0.075	0.129	10256614	39.579	0.5	0.027	0.131	0.131	0.4	0.75	0.086	0.393
46	0.505	14067450	0.643	9049832.5	2813490	50	56270	55.4	0.05	9100	124	477	0.039	0.048	300	0.02	0.039	0.069	0.116	5017617	44.585	0.5	0.039	0.127	0.127	0.4	0.75	0.078	0.381
47	1.544	43055568	0.740	31873114	8611114	50	172222	67.5	0.05	8400	130	650	0.062	0.047	300	0.02	0.062	0.061	0.108	11182454	32.465	0.5	0.062	0.098	0.098	0.4	0.75	0.072	0.294
48	2.054	57257856	0.790	45208194	11451571	50	229031	73.7	0.05	14700	78	310	0.016	0.067	300	0.02	0.016	0.085	0.152	12049662	26.306	0.5	0.016	0.123	0.123	0.4	0.75	0.101	0.368

1 Source: Nolte & Assoc 2000, Table 1 gives street area as 20% of total area for range of urban land uses
2 Source: SCVWD Hydrology Procedures, 1998. Typical average street width = 50 ft.
3 Source: SCVWD Hydrology Procedures, 1998. Lst=Ast/Wst
4 Source: SCVWD Hydrology Procedures, 1998. Li=Ai-Ast/2Lst
5 Source: SCVWD Hydrology Procedures, 1998. Kerby-Hathaway Tc equation
6 Source: Nolte & Assoc 2000, average gutter length between catch basins = 300 ft.
7 Source: SCVWD Hydrology Procedures, 1998. TCimp= TCi + TCcb
8 Source: SCVWD Hydrology Procedures, 1998. TCperv= TCimp + TCp
9 Source: SCVWD Hydrology Procedures, 1998 (p.26, range); Nolte & Assoc 2000 uses 0.4 and 0.75.

Note: All Tc and R values less than 0.1 are rounded up to 0.1 for input into HEC-HMS.

Table 4-8: Time of Concentration and Storage Coefficient, Thompson Creek, Future Conditions, Rural and External Pervious Areas

	Subbasin	Length (ft)	Min. Elev. (ft)	Max. Elev. (ft)	Slope (ft/ft)	n		Tc (hr)	R / (Tc + R)	R
Rural	2	19200	310	2203	0.0986	0.070		0.701	0.750	2.103
Subbasins	3	10500	523	2449	0.1834	0.070		0.456	0.750	1.369
	4	10900	480	2403	0.1764	0.070		0.469	0.750	1.406
	7	3200	126	134	0.0025	0.070		0.716	0.750	2.149
	9	2300	125	134	0.0039	0.070		0.552	0.750	1.656
	10	9800	495	2418	0.1962	0.070		0.435	0.750	1.304
	11	16100	259	2384	0.1320	0.070		0.603	0.750	1.808
	12	10200	243	1842	0.1568	0.070		0.467	0.750	1.401
	21	14700	448	2273	0.1241	0.070		0.586	0.750	1.757
	22	17700	291	2269	0.1118	0.070		0.655	0.750	1.965
	26	11300	487	2341	0.1641	0.070		0.485	0.750	1.454
	27	12800	664	2153	0.1163	0.070		0.557	0.750	1.672
	29	9200	845	2480	0.1777	0.070		0.432	0.750	1.296
	32	8300	689	1789	0.1325	0.070		0.441	0.750	1.323
	33	9400	664	2246	0.1683	0.070		0.442	0.750	1.326
	34	10900	644	2491	0.1694	0.070		0.473	0.750	1.419
	38	6000	604	1417	0.1355	0.070		0.377	0.750	1.130
	40	4200	303	503	0.0476	0.070		0.407	0.750	1.222
	49	4300	552	1050	0.1158	0.070		0.334	0.750	1.002
							plus Urban Tc-Impervious			
External	6	1900	320	609	0.1521	0.070	0.129	0.343	0.750	1.028
Pervious	8	7700	246	1342	0.1423	0.070	0.129	0.548	0.750	1.643
Subbasins	13	3400	233	502	0.0791	0.070	0.152	0.479	0.750	1.438
	14	5000	279	854	0.1150	0.070	0.116	0.476	0.750	1.427
	15	3500	212	943	0.2089	0.070	0.108	0.372	0.750	1.117
	20	5200	506	920	0.0796	0.070	0.109	0.508	0.750	1.523
	24	4600	441	1182	0.1611	0.070	0.122	0.441	0.750	1.323
	37	3500	616	963	0.0991	0.070	0.121	0.435	0.750	1.306
	43	1300	520	823	0.2331	0.070	0.129	0.291	0.750	0.872

Notes:

Overland flow Manning's roughness coefficient 0.07 represents "sparse vegetation" (SCVWD, 1998).

The ratio of R / (Tc + R) lies between .5 and .9 for pervious and rural areas (SCVWD, 1998);

a ratio of 0.75 was chosen for rural hillside open space.

Table 4-9: Time of Concentration and Storage Coefficient, Ross Creek, Pre-Urban Conditions

Subbasin[1]	Tc (hr)	R / (Tc + R)	R
0	0.723	0.99	56
1	0.727	0.99	53
2	0.871	0.89	7
3	0.820	0.75	2.4
4	1.079	0.98	59
5	0.675	0.98	44
6	0.500	0.99	56
7	0.652	0.99	53
8	0.794	0.99	55
9	0.500	0.99	60
10	0.697	0.99	55
11	0.500	0.81	2.2
12	0.544	0.99	44
13	0.573	0.84	3
14	0.573	0.99	39
15	0.573	0.96	14.5
16	0.573	0.96	14.7
17	0.573	0.92	7

Notes:

1) All Pre-urban subbasins are considered rural

Table 4-10: Time of Concentration and Storage Coefficient, Ross Creek, Existing Conditions, Urban Areas

	Impervious			Pervious		
Subbasin	Tc (hr)	R / (Tc + R)	R	Tc (hr)	R / (Tc + R)	R
0	0.885	0.22	0.25	0.916	0.97	26.50
1	0.817	0.77	2.72	0.870	0.96	21.58
2	0.837	0.43	0.62	0.936	0.96	22.06
3	0.778	0.27	0.29	0.861	0.94	13.70
4	0.774	0.58	1.05	0.961	0.96	25.30
5	0.665	0.28	0.26	0.799	0.96	20.76
6	0.500	0.93	7.12	0.674	0.98	29.75
8	0.707	0.91	7.08	1.055	0.97	29.78
9	0.569	0.41	0.39	0.726	0.85	4.27
11	0.716	0.98	29.01	0.895	0.97	27.50
13	0.529	0.32	0.25	0.787	0.24	0.25

Table 4-11: Time of Concentration and Storage Coefficient, Ross Creek, Existing Conditions, Rural and External Pervious Areas

Subbasin	Type[1]	Tc (hr)	R / (Tc + R)	R
12	R	0.544	0.98	28.55
14	R	0.500	0.34	0.26
15	R	0.500	0.71	1.24
16	R	0.500	0.98	27.63
17	R	0.500	0.98	29.97
7	E	0.752	0.97	23.11
10	E	0.819	0.92	9.54

Notes:

1) R = Rural; E = Pervious urban external of storm drain system

Table 4-12: Time of Concentration and Storage Coefficient, Ross Creek, Future Conditions, Urban Areas

	Impervious			Pervious		
Subbasin	Tc (hr)	R / (Tc + R)	R	Tc (hr)	R / (Tc + R)	R
0	0.885	0.22	0.25	0.916	0.97	26.50
1	0.817	0.77	2.72	0.870	0.96	21.58
2	0.837	0.43	0.62	0.936	0.96	22.06
3	0.778	0.27	0.29	0.861	0.94	13.70
4	0.774	0.58	1.05	0.961	0.96	25.30
5	0.665	0.28	0.26	0.799	0.96	20.76
6	0.500	0.93	7.12	0.674	0.98	29.75
8	0.707	0.91	7.08	1.055	0.97	29.78
9	0.569	0.41	0.39	0.726	0.85	4.27
11	0.716	0.98	29.01	0.895	0.97	27.50
13	0.529	0.32	0.25	0.787	0.24	0.25

Table 4-13: Time of Concentration and Storage Coefficient, Ross Creek, Future Conditions, Rural and External Pervious Areas

Subbasin	Type[1]	Tc (hr)	R / (Tc + R)	R
12	R	0.544	0.98	28.55
14	R	0.500	0.34	0.26
15	R	0.500	0.71	1.24
16	R	0.500	0.98	27.63
17	R	0.500	0.98	29.97
7	E	0.752	0.97	23.11
10	E	0.819	0.92	9.54

Notes:

1) R = Rural; E = Pervious urban external of storm drain system

Table 4-14: Clark Unit Hydrograph Parameter Estimation: Storage Routing for Thompson Creek

STO i.d.	1		2		3		4		5		6		7		8		9		10		11	
area, ac (1)	46		69		26		313		38		159		184		532		250		73		920	
Levees? Y/N (2)	N		N		N		N		N		N		N		N		N		N		Y	
	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	0.2	3.3	0.3	5.0	0.1	1.9	1.2	22.5	0.1	2.8	0.6	11.4	0.7	13.2	2.1	23.3	1.0	18.0	0.3	5.2	3.6	40.3
	0.5	10.8	0.8	16.2	0.3	6.0	3.7	73.4	0.4	9.0	1.9	37.2	2.2	43.1	6.2	58.2	2.9	58.6	0.9	17.1	10.8	100.6
	0.7	12.9	1.1	19.5	0.4	7.3	4.9	88.1	0.6	10.8	2.5	44.7	2.9	51.7	8.3	66.5	3.9	70.3	1.1	20.5	14.4	115.0
	0.9	14.7	1.4	22.2	0.5	8.3	6.1	100.4	0.7	12.3	3.1	50.9	3.6	58.9	10.4	70.6	4.9	80.1	1.4	23.4	18.0	122.2
	1.4	17.2	2.2	25.9	0.8	9.7	9.8	117.5	1.2	14.4	5.0	59.6	5.7	69.0	16.6	103.9	7.8	93.7	2.3	27.3	28.8	126.5
	2.2	20.1	3.2	30.3	1.2	11.3	14.7	137.1	1.8	16.8	7.4	69.5	8.6	80.5	24.9	128.8	11.7	109.3	3.4	31.9	43.1	130.8
	12.9	20.1	19.5	30.3	7.3	11.3	88.1	137.1	10.8	16.8	44.7	69.5	51.7	80.5	149.6	128.8	70.3	109.3	20.5	31.9	258.8	130.8

12		13		14		15		16		17		18		19		20		21		22	
1202		999		1405		195		1308		517		465		1526		2543		1476		1559	
Y		Y		Y		Y		Y		Y		Y		Y		Y		Y		Y	
vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)	vol(ac-ft)	Q(cfs)
0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4.7	52.6	3.9	43.7	5.5	61.5	0.8	14.0	5.1	57.2	2.0	22.6	1.8	20.3	6.0	66.8	9.9	111.2	5.8	64.6	6.1	68.2
14.1	131.5	11.7	109.3	16.5	153.7	2.3	45.7	15.3	143.0	6.1	56.6	5.4	50.9	17.9	166.9	29.8	278.1	17.3	161.5	18.3	170.5
18.8	150.2	15.6	124.9	22.0	175.6	3.0	54.9	20.4	163.5	8.1	64.7	7.3	58.1	23.8	190.8	39.7	317.8	23.1	184.5	24.4	194.9
23.5	159.6	19.5	132.7	27.4	186.6	3.8	62.5	25.5	173.7	10.1	68.7	9.1	61.8	29.8	202.7	49.7	337.7	28.8	196.1	30.4	207.0
37.6	165.3	31.2	137.4	43.9	193.2	6.1	64.0	40.9	179.8	16.2	71.1	14.5	63.9	47.7	209.9	79.5	349.6	46.1	203.0	48.7	214.3
56.3	170.9	46.9	142.1	65.9	199.8	9.1	67.0	61.3	186.0	24.2	73.6	21.8	66.1	71.5	217.0	119.2	361.5	69.2	209.9	73.1	221.6
338.0	170.9	281.1	142.1	395.2	199.8	54.9	67.0	367.8	186.0	145.5	73.6	130.8	66.1	429.2	217.0	715.1	361.5	415.2	209.9	438.4	221.6

Unitized Storage-Discharge Relationships (3)

condition								
1 A > 400 ac w/o levees								
vol (ac-ft/sq mi)	0	2.5	7.5	10	12.5	20	30	180
Q (cfs/sq mi)	0	28	70	80	85	125	155	155
2 A > 400 ac w/ levees								
vol (ac-ft/sq mi)	0	2.5	7.5	10	12.5	20	30	180
Q (cfs/sq mi)	0	28	70	80	85	88	91	91
3 A < 400 ac w/o levees								
vol (ac-ft/sq mi)	0	2.5	7.5	10	12.5	20	30	180
Q (cfs/sq mi)	0	46	150	180	205	240	280	280
4 A < 400 ac w/ levees								
vol (ac-ft/sq mi)	0	2.5	7.5	10	12.5	20	30	180
Q (cfs/sq mi)	0	46	150	180	205	210	220	220

E = Pervious "External"
I = Impervious
P = Pervious

(1) "Total area" reflects sum of Pervious, Impervious and Undeveloped areas contributing to storm drains for each STO module

(2) Areas upstream of Yerba Buena were assigned "w/o levee" status, downstream of Yerba Buena assigned "w/levee" status following general guidance in SCVWD, 1998

(3) Tables derived from plot in SCVWD, 1998 Fig 7

Table 4-15: HEC-HMS Reach Routing, Thompson Creek, Pre-Urban Conditions

HMS ID	Creek	Connection	Shape	Length, ft	Slope, ft/ft	Width or Diameter, ft	H:V	Manning's n	Source Information (width, diameter, H:V, & n)
1	Thompson	27TR - J-1	Prism	375	0.02670	4.00	0.80	0.040	A
2	Thompson	J-1 - J-2	Prism	2188	0.02513	7.00	0.40	0.045	B
3	Hawk	33HR - J-2	Prism	1166	0.04290	4.00	0.80	0.040	A
4	Thompson	J-2 - J-3	Prism	2559	0.01563	7.00	0.40	0.045	B
5	Misery	34MR - J-3	Prism	2237	0.03800	4.00	0.80	0.040	A
6	Thompson	J-3 - J-5	Prism	5420	0.01753	7.00	0.40	0.045	B
7		38CR - J-4	Prism	2600	0.02671	4.00	0.80	0.035	A, B
8	Cribari	29CR - J-4	Prism	7400	0.04912	6.00	0.80	0.035	A, B
9	Cribari	J-4 - J-5	Prism	1640	0.03173	7.00	0.80	0.035	A, B
10	Thompson	J-5 - J-6	Prism	3300	0.01667	7.00	1.00	0.050	B
11	Thompson	J-6 - J-7	Prism	6030	0.01658	9.00	0.10	0.050	B
12	Yerba Buena	21YR - J-7	Prism	4835	0.02792	11.00	2.10	0.030	B
13	Thompson	J-7 - J-9	Prism	2871	0.01045	9.00	1.00	0.040	A, B
14	Evergreen	J-8 - J-9	Prism	414	0.03000	8.00	2.00	0.030	A, B
16	Thompson	J9 - J-10	Prism	6366	0.00943	12.00	1.04	0.035	B
17		26WR - J-10	Prism	9700	0.03062	10.00	0.80	0.035	A, B
18	Fowler	3WR - J-10	Prism	9700	0.03062	10.00	0.80	0.035	A, B
19	Thompson	J-10 - J-11	Prism	3552	0.01056	13.00	1.64	0.045	B
20	Quimby	4QR - J-11	Prism	9700	0.03267	10.00	0.80	0.035	A, B
21	Thompson	J-11 - J-12	Prism	5837	0.00728	14.00	0.61	0.030	B
22	Norwood	10NR - J-12	Prism	14000	0.02843	10.00	0.80	0.035	A, B
23	Thompson	J-12 - J-13	Prism	2634	0.00627	20.48	1.00	0.031	B, D
24	Thompson	J-13 - J-14	Prism	4217	0.00363	37.33	1.00	0.033	B, D
25	Flint	11FR - J-14	Prism	6400	0.01927	10.00	0.80	0.035	A, B
26	Lower Silver	J-14 - J-15	Prism	4379	0.00200	47.70	2.00	0.035	D
27	South Babb	22BR - J-15	Prism	8454	0.01881	15.00	1.50	0.030	A, B
28	Lower Silver	J-15 - J-16	Prism	3235	0.00200	37.00	1.23	0.035	D
29	North Babb	STO-19 - J-16	Prism	1307	0.00612	10.00	1.00	0.025	A, B
30	Lower Silver	J-16 - J-17	Prism	12041	0.00200	41.00	0.75	0.035	D
31	Lower Silver	J-17 - J-18	Prism	1100	0.00200	46.00	0.01	0.035	D
32	Miguelita	12GR - J-18	Prism	14200	0.00853	10.00	0.80	0.035	A, B, D
33	Lower Silver	J-18 - J-19	Prism	2020	0.00200	65.00	0.96	0.035	D

Lengths approximated by querying WLA creek and storm drain GIS layers.
Slopes approximated using length estimates and USGS DEM elevations.

Past reach lengths validated or changed based on 1964 arial photographs (Pafford & Associates, suveyors).
Existing storm drains throughout watershed NOT present in 1964; historic channel charactersitcs estimated for these reaches.

Source information used to approximate width, diameter, H:V, and Manning's n:
- A - Nolte (2000)
- B - PWA and Balance Hydrologics survey (2002)
- D - Lower Silver HEC-RAS model

Note: there is no R-15

Table 4-16: HEC-HMS Reach Routing, Thompson Creek, Existing and Future Conditions

HMS ID	Creek	Connection	Shape	Length, ft	Slope, ft/ft	Width or Diameter, ft	H:V	Manning's n	Source Information (width, diameter, H:V, & n)
1	Thompson	27TR - J-1	Prism	375	0.02670	4.00	0.80	0.040	A
2	Thompson	J-1 - J-2	Prism	2188	0.02513	7.00	0.40	0.045	B
3	Hawk	33HR - J-2	Prism	1166	0.04290	4.00	0.80	0.040	A
4	Thompson	J-2 - J-3	Prism	2559	0.01563	7.00	0.40	0.045	B
5	Misery	34MR - J-3	Prism	2237	0.03800	4.00	0.80	0.040	A
6	Thompson	J-3 - J-5	Prism	5420	0.01753	7.00	0.40	0.045	B
7		38CR - J-4	Circle	2991	0.02671	3.02		0.013	C
8	Cribari	29CR - J-4	Circle	6658	0.04912	4.38		0.014	C
9	Cribari	J-4 - J-5	Circle	1640	0.03173	5.50		0.013	C
10	Thompson	J-5 - J-6	Prism	3300	0.01667	7.00	1.00	0.050	B
11	Thompson	J-6 - J-7	Prism	6030	0.01658	9.00	0.10	0.050	B
12	Yerba Buena	21YR - J-7	Prism	4835	0.02792	11.00	2.10	0.030	B
13	Thompson	J-7 - J-9	Prism	2871	0.01045	9.00	1.00	0.040	A, B
14	Evergreen	J-8 - J-9	Prism	414	0.03000	8.00	2.00	0.030	A, B
16	Thompson	J9 - J-10	Prism	6366	0.00943	12.00	1.04	0.035	B
17		26WR - J-10	Circle	9178	0.03062	6.65		0.013	C
18	Fowler	3WR - J-10	Circle	9178	0.03062	6.65		0.013	C
19	Thompson	J-10 - J-11	Prism	3552	0.01056	13.00	1.64	0.045	B
20	Quimby	4QR - J-11	Circle	9762	0.03267	6.13		0.013	C
21	Thompson	J-11 - J-12	Prism	5837	0.00728	14.00	0.61	0.030	B
22	Norwood	10NR - J-12	Circle	12371	0.02843	6.34		0.013	C
23	Thompson	J-12 - J-13	Prism	2634	0.00627	20.48	1.00	0.031	B, D
24	Thompson	J-13 - J-14	Prism	4217	0.00363	37.33	1.00	0.033	B, D
25	Flint	11FR - J-14	Circle	6538	0.01927	6.87		0.013	C
26	Lower Silver	J-14 - J-15	Prism	4379	0.00200	47.70	2.00	0.035	D
27	South Babb	22BR - J-15	Prism	8454	0.01881	15.00	1.50	0.030	A, B
28	Lower Silver	J-15 - J-16	Prism	3235	0.00200	37.00	1.23	0.035	D
29	North Babb	STO-19 - J-16	Prism	1307	0.00612	10.00	1.00	0.025	A, B
30	Lower Silver	J-16 - J-17	Prism	12041	0.00200	41.00	0.75	0.035	D
31	Lower Silver	J-17 - J-18	Prism	1100	0.00200	46.00	0.01	0.035	D
32	Miguelita	12GR - J-18	Circle	15570	0.00853	6.50		0.013	C
33	Lower Silver	J-18 - J-19	Prism	2020	0.00200	65.00	0.96	0.035	D

Lengths approximated by querying WLA creek and storm drain GIS layers.
Slopes approximated using length estimates and USGS DEM elevations.

Source information used to approximate width, diameter, H:V, and Manning's n:

A - Nolte (2000)
B - PWA and Balance Hydrologics survey (2002)
C - Storm drain analysis
D - Lower Silver HEC-RAS model

Note: there is no R-15

Table 4-17: HEC-HMS Reach Routing, Ross Creek, Pre-Urban Conditions

HMS ID	Connection	Shape	Length, ft	Slope, ft/ft	Width, ft	H:V	Manning's n
0	J0 - J1	Prism	2600	0.0027	12.0	1.4	0.040
1	J1 - J2	Prism	5200	0.0038	12.0	1.4	0.040
2	J2 - J3	Prism	4200	0.0060	12.0	1.4	0.045
3	J3 - SB6	Prism	6300	0.0071	10.0	1.1	0.060
4	J3 - J4	Prism	6100	0.0082	12.0	1.1	0.050
5	J4 - J5	Prism	7900	0.0089	12.0	1.1	0.060
6	J5 - J6	Prism	900	0.0111	12.0	1.1	0.070
7	J6 - SB14	Prism	5100	0.0108	6.0	1.6	0.055
8	J6b - J7	Prism	5600	0.0254	6.0	1.6	0.055
8b	J6 - J6b	Prism	100	0.0254	6.0	1.6	0.055
9	J7 - SB17	Prism	1400	0.0571	3.0	2.2	0.045

Table 4-18: HEC-HMS Reach Routing, Ross Creek, Existing and Future Conditions

HMS ID	Connection	Shape	Length, ft	Slope, ft/ft	Width, ft	H:V	Manning's n
0	J0 - J1	Prism	2600	0.0027	12.0	1.4	0.040
1	J1 - J2	Prism	5200	0.0038	12.0	1.4	0.040
2	J2 - J3	Prism	4200	0.0060	12.0	1.4	0.045
3	J3 - SB6	Prism	6300	0.0071	10.0	1.1	0.060
4	J3 - J4	Prism	6100	0.0082	12.0	1.1	0.050
5	J4 - J5	Prism	7900	0.0089	12.0	1.1	0.060
6	J5 - J6	Prism	900	0.0111	12.0	1.1	0.070
7	J6 - SB14	Prism	5100	0.0108	6.0	1.6	0.055
8	J6b - J7	Prism	5700	0.0254	6.0	1.6	0.055
8b	J6 - J6b	Prism	100	0.0254	6.0	1.6	0.055
9	J7 - SB17	Prism	1400	0.0571	3.0	2.2	0.045

Table 4-19: Soil Moisture Accounting Parameter Initialization

Input Parameter	Source	Method
Maximum Canopy Interception Storage	Land use layer (GIS) from William Lettis & Assoc. (WLA) Vegetation-interception relationships from literature Mean Annual Precipitation (MAP) data from SCVWD	1. Interception storage assigned for each vegetation type present using published data from similar environments (% of MAP) 2. MAP assigned for each drainage basin 3. Interception converted to depth using precipitation depth for frequent design storm from SCVWD 1998 (1"); depths verified vs. values from literature
Maximum Surface Depression Storage	Land use data (GIS) from WLA HEC-HMS Manual	1. Surface storage depression value assigned per land use (GIS) using HEC-HMS manual guidance 2. Drainage area surface storage determined as a weighted average of land use using GIS data
Maximum Infiltration Rate	Soil data from SCVWD database Soil parameters estimated from the literature (Rawls et al., 1982)	1. The initial maximum infiltration rate was estimated for each of the Hydrologic Soil Groups present (B, C, and D) using the Green-Ampt equation and soil parameter estimates from the literature. 2. All infiltration rates were then scaled up during model verification
Maximum Percolation Rate	Soil data from SCVWD database Saturated hydraulic conductivity (K_s) for representative soils estimated from literature	1. The Maximum Percolation Rate was set equal to saturated hydraulic conductivity (K_s)
Maximum Soil Profile Storage	Soil data from SCVWD database Soil profile depth from Santa Clara County Soil Survey (NRCS) and effective porosity from Rawls et al, 1982	1. Soil profile storage for each soil type taken from SCVWD data (in GIS) as a product of effective soil depth (root zone depth) and effective porosity. 2. Weighted average Maximum Soil Profile Storage determined for each drainage area using SCVWD data in GIS
Tension (Unsaturated) Zone Storage	Soil data from SCVWD database HEC-HMS manual and Santa Clara County Soil Survey (NRCS)	1. Tension zone storage estimated as Available Water holding Capacity (A.W.C) from the County Soil Survey
Maximum Groundwater Layer Storage	N/A	1. Following guidance from the literature, groundwater storage and percolation rate were not considered in this assessment due to the lack of base flow in the creeks

Table 4-20: Simulated and Measured Thompson Creek Discharge Volumes and Precipitation, Sping 2003

	Location	Thompson Creek at Quimby Road		Yerba Buena Creek watershed	
Event	HEC-HMS location Gage name	J-11 TCQR gage	% of precip volume	STO-8 + R-12 YBSF gage	% of precip volume
	Drainage area, sq-mi	17.37		2.58	
	Drainage area, ac	11115		1650	
Gage time period		Jan 3 (12:00) to Apr 9 (24:00)		Jan 8 (17:00) to Mar 18 (14:00)	
	Precipitation, in	3.685		3.024	
	Precipitation, ac-ft	3412.9		415.8	
	HEC Continuous., ac-ft	168.8	4.9	14.7	3.5
	Gage, ac-ft	163.3	4.8	2.4	0.6
January 9-10	Precipitation, in	0.564		0.564	
	Precipitation, ac-ft	522.4		77.5	
	HEC Event-based, ac-ft	51.7	9.9	5.0	6.5
	HEC Continuous, ac-ft	35.2	6.7	3.1	3.9
	Gage, ac-ft	33.6	6.4	1.1	1.4
February 16	Precipitation, in	0.420		0.420	
	Precipitation, ac-ft	388.8		57.7	
	HEC Event-based, ac-ft	32.9	8.5	3.1	5.4
	HEC Continuous, ac-ft	14.6	3.8	1.1	1.9
	Gage, ac-ft	19.8	5.1	0.2	0.3
February 25-27	Precipitation, in	0.950		0.950	
	Precipitation, ac-ft	879.7		130.6	
	HEC Event-based, ac-ft	119.7	13.6	12.9	9.9
	HEC Continuous, ac-ft	76.2	8.7	6.9	5.3
	Gage, ac-ft	40.7	4.6	0.4	0.3
March 15-16	Precipitation, in	0.657		0.657	
	Precipitation, ac-ft	608.9		90.4	
	HEC Event-based, ac-ft	64.1	10.5	6.4	7.1
	HEC Continuous, ac-ft	41.5	6.8	3.6	4.0
	Gage, ac-ft	39.2	6.4	0.5	0.5

Table 4-21: Thompson Creek, Event-Based Design Storm Results

Return Period (years)	Q-peak, cfs						V-total, ac-ft					
	2	5	10	25	50	100	2	5	10	25	50	100
Pre-Urban SCS (24-hr balanced)												
Junction J-3	127.2	311.4	454.7	643.0	781.7	920.1	63.6	155.0	226.7	324.6	400.2	477.2
J-5	189.1	451.4	653.4	917.7	1112.6	1305.1	99.2	235.8	342.2	486.5	597.8	710.7
J-7	274.2	650.7	937.3	1313.7	1592.1	1868.8	154.1	365.9	530.8	754.2	926.6	1101.5
J-9	321.5	763.6	1098.8	1539.1	1866.4	2188.7	181.4	432.2	627.5	892.6	1097.2	1304.7
J-10	438.1	1032.6	1479.7	2064.1	2496.4	2922.3	252.2	595.2	861.5	1221.8	1499.5	1781.0
J-11	529.6	1229.4	1751.6	2430.6	2933.6	3428.3	316.6	735.9	1059.5	1496.0	1831.7	2171.6
J-12	660.4	1494.2	2110.2	2909.3	3499.4	4081.0	434.0	980.2	1397.3	1957.0	2385.7	2818.9
Subbasin SB-27	55.7	138.7	203.1	289.0	353.0	416.2	28.0	69.7	102.6	147.6	182.5	218.0
SB-21	57.2	137.6	198.8	279.7	340.3	400.0	29.2	70.9	103.5	147.8	182.2	217.0
SB-41	13.7	32.6	46.7	65.0	78.7	92.3	12.2	29.9	43.8	62.6	77.2	92.0
Reach-12	57.2	137.6	198.7	279.6	340.0	399.7	29.2	70.9	103.5	147.8	182.2	217.0
Total Yuerba Buena	69.2	166.4	239.1	336.5	409.8	482.8	41.5	100.8	147.3	210.5	259.3	309.0
SB-39	10.4	19.3	25.4	33.0	38.5	43.8	8.2	16.2	21.9	29.3	34.8	40.3
Exsiting SCS (24-hr balanced)												
Junction J-3	136.4	335.6	486.0	679.0	820.6	960.3	66.8	158.8	230.9	329.0	404.8	481.9
J-5	266.6	591.9	812.3	1082.9	1278.9	1473.4	129.2	276.2	388.6	539.7	655.6	772.9
J-7	442.3	918.2	1244.0	1630.4	1907.2	2168.0	222.8	456.4	633.2	869.7	1050.6	1233.1
J-9	495.5	1045.4	1423.2	1876.4	2199.0	2510.1	252.8	525.1	731.9	1009.1	1221.4	1435.8
J-10	712.9	1389.6	1856.2	2420.0	2825.7	3215.9	378.2	754.8	1038.2	1416.2	1704.8	1995.9
J-11	924.1	1671.3	2182.7	2798.4	3244.6	3670.0	498.9	965.0	1312.2	1772.8	2123.3	2476.0
J-12	1306.6	2129.7	2683.7	3351.7	3836.4	4298.3	756.7	1383.1	1840.7	2441.6	2895.9	3351.4
Subbasin SB-27	55.7	138.7	203.1	289.0	353.0	416.2	28.0	69.7	102.6	147.6	182.5	218.0
SB-20	22.8	42.1	55.1	70.7	81.8	92.6	10.0	19.2	25.8	34.3	40.6	46.9
SB-21	57.2	137.6	198.8	279.7	340.3	400.0	29.2	70.9	103.5	147.8	182.2	217.0
SB-41-Impervious	140.5	196.3	231.7	274.4	304.5	334.5	22.9	34.1	41.4	50.3	56.8	63.1
SB-41-Pervious	6.4	28.2	45.4	68.0	85.1	102.3	2.3	6.8	10.5	15.7	19.8	24.1
Reach-12	57.2	137.6	198.7	279.6	340.0	399.7	29.2	70.9	103.5	147.8	182.2	217.0
STO-8	60.0	83.9	104.3	121.8	128.8	128.8	35.3	60.1	77.7	100.3	117.2	134.0
Total Yuerba Buena	115.8	221.3	301.3	396.5	467.9	528.5	64.5	131.0	181.2	248.2	299.4	351.0
SB-39-Impervious	70.4	98.4	116.1	137.5	152.5	167.5	11.5	17.1	20.8	25.3	28.5	31.7
SB-39-Pervious	14.6	31.4	43.1	57.5	68.0	78.3	3.5	7.5	10.4	14.3	17.3	20.3
Total SB 39	82.9	126.0	154.1	188.4	212.8	237.0	15.0	24.6	31.2	39.6	45.8	51.9
Future SCS (24-hr balanced)												
Junction J-3	181.7	404.5	562.1	761.5	907.5	1050.0	86.5	189.7	268.1	373.2	453.5	534.6
J-5	356.3	699.9	921.9	1199.6	1401.4	1595.9	165.7	329.4	451.1	612.3	734.6	857.5
J-7	586.0	1098.6	1418.8	1810.1	2082.1	2342.6	293.1	555.7	748.3	1001.7	1193.2	1385.1
J-9	696.7	1297.8	1676.1	2137.2	2459.4	2769.9	350.7	664.2	893.6	1194.8	1422.4	1650.1
J-10	962.9	1706.8	2179.3	2757.2	3164.2	3555.2	504.0	935.2	1248.5	1658.4	1967.3	2276.1
J-11	1187.7	2001.9	2519.3	3152.8	3595.5	4025.6	644.7	1172.9	1554.0	2050.7	2424.0	2796.8
J-12	1584.3	2476.2	3036.9	3724.3	4204.8	4673.7	922.6	1618.2	2113.4	2754.5	3234.0	3711.6
Subbasin SB-27	71.5	163.3	232.2	321.9	387.5	452.3	35.1	81.3	116.8	164.8	201.5	238.8
SB-20	47.6	67.5	79.8	94.2	104.3	114.0	22.0	34.1	42.0	51.8	58.9	65.8
SB-21	79.1	170.3	236.8	322.5	385.1	446.1	39.3	87.0	123.1	171.2	208.0	244.9
SB-41-Impervious	161.1	225.2	265.8	314.8	349.2	383.6	26.4	39.3	47.6	57.9	65.4	72.6
SB-41-Pervious	5.7	25.0	40.1	60.0	74.9	90.0	2.0	5.7	8.9	13.3	16.8	20.3
Reach-12	79.0	170.3	236.8	322.2	384.8	445.8	39.4	87.0	123.1	171.2	208.0	245.0
STO-8	72.6	104.7	118.7	128.8	128.8	128.8	50.3	79.1	98.5	123.0	141.0	158.8
Total Yuerba Buena	151.6	274.1	352.0	451.0	513.6	574.6	89.7	166.1	221.5	294.2	349.0	403.7
SB-39-Impervious	94.9	132.6	156.5	185.3	205.6	225.8	15.7	23.3	28.3	34.4	38.8	43.1
SB-39-Pervious	10.9	23.2	31.7	42.3	50.0	57.5	2.4	5.0	7.0	9.6	11.6	13.6
Total SB-39	104.6	153.7	185.5	224.1	251.5	278.8	18.0	28.3	35.3	44.0	50.4	56.7
	2	5	10	20	50	100						
Historical Flood Quantile (SCVWD, 1998)												
Junction J-3	171.9	310.6	426.9	553.5	750.3	923.0						
J-5	252.1	460.4	639.0	837.0	1149.7	1429.2						
J-7	339.7	640.9	907.6	1209.8	1697.3	2142.5						
J-12	903.1	1583.0	2172.1	2829.0	3880.3	4838.1						

Table 4-22: Pre-Urban Calibration - Period from 10/1/1950 - 3/31/1952

	Volume (ac-ft)	Average Q (cfs)	Model Results' RMS Error	Percent Error in Volume
Gage 21	2,500	2.3	30	12%
Gage 21 - Flows < 1 cfs	2,350	2.2	31	7%
Model Results @ J-5	2,190	2.0	-	-

Table 4-23: Pre-Urban Calibration - Period from 10/1/1956 - 5/1/1958

	Volume (ac-ft)	Average Q (cfs)	Model Results' RMS Error	Percent Error in Volume
Gage 51	3,260	2.9	95	34%
Gage 51 - Flows < 1 cfs	2,130	2.0	46	-1%
Model Results @ J-1	2,160	1.9	-	-

Table 4-24: Existing Condition Calibration - Period from 11/1/200 - 3/31/2003

	Volume (ac-ft)	Average Q (cfs)	Model Results' RMS Error	Percent Error in Volume
Gage 21	2,810	3.0	44	61%
Gage 21 - Flows < 1 cfs	2,510	2.6	47	56%
Model Results @ J-5	1,100	1.2	-	-

Table 4-25: Existing Condition Calibration - Period from 11/1/200 - 3/31/2003

	Volume (ac-ft)	Average Q (cfs)	Model Results' RMS Error	Percent Error in Volume
Gage 51	5,080	5.0	126	3%
Gage 51 - Flows < 1 cfs	4,730	4.6	130	-4%
Model Results @ J-1	4,910	4.9	-	-

Appendix A

Appendix A **–Logistic Regression**

Appendix B **–Technical Memorandum #4**

–Technical Memorandum #5

–Technical Memorandum #7

Appendix A
Logistic Regression for Stream Stability

This appendix briefly describes how logistic regression is applied to stream stability issues. An interested reader is referred to *Statistical Methods in Water Resources* (Helsel D.R. and Hirsch R.M., 1992).

Introduction

One of the goals of the HMP project is to predict stream stability based on measurable variables such as velocity, shear stress, work done to the streams, and erosion potential (EP). Stream stability, categorized into stable or unstable, is a qualitative and discrete variable, whereas the measurable variables are quantitative and continuous. Logistic regression, also called **logit** regression, is an appropriate statistical tool for this situation.

Within the framework of logistic regression used in this project, stream stability is characterized as a binary response variable, expressing itself as either 0 – stable, or 1 – unstable, and the measurable variables as continuous explanatory variables. Given a value of one of the explanatory variables, logistic regression provides the probability of the stream stability being stable or unstable. Note that, although multiple logistic regression (i.e., multiple explanatory variables) is also possible, in this project, only simple logistic regression (i.e., one explanatory variables) will be considered.

Important Formulae

Let p be the probability of a stream being unstable. The valid range of value of p is between 0 and 1. Odds ratio is defined as:

$$Odds\ ratio = \frac{p}{1-p}$$

The natural log of the odds ratio is called **logit**. Thus,

$$Logit = \ln\left[\frac{p}{1-p}\right]$$

At this point, p, whose range is 0 to 1, has been transformed into the logit, which is continuous and unbounded, and thus applicable to linear regression. Logistic regression, or **logit** regression, is essentially simple linear regression relating a continuous explanatory variable to the stream stability logit. In other words, it seeks optimal values of the intercept (b_o) and slope coefficient (b) in the following equation:

$$\ln\left[\frac{p}{1-p}\right] = b_o + bx$$

where x is a continuous explanatory variable (e.g., velocity, shear stress, work done to the streams, or erosion potential (EP)).

Therefore, p can be predicted for a given value of the explanatory variable x, and is expressed as:

$$p = \frac{\exp(b_o + bx)}{1 + \exp(b_o + bx)}$$

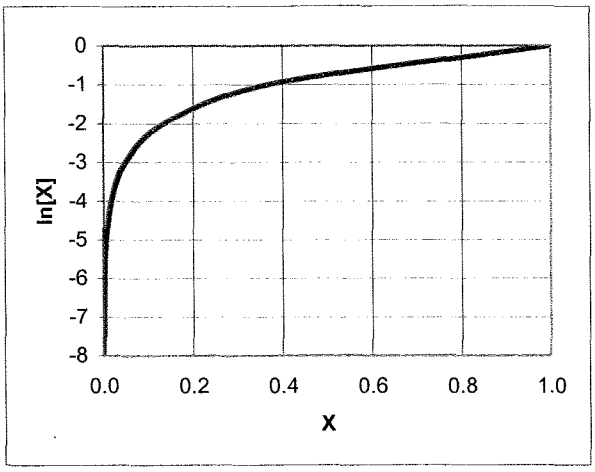
Optimal values of b_o and b are obtained when log likelihood (l) is maximized (see the equation below). Note that l is always negative; maximizing l is thus bringing it closet to zero.

$$l = \sum_{i=1}^{n} (y_i \cdot \ln[p_i] + (1 - y_i) \cdot \ln[1 - p_i])$$

The following is the rationale behind the use of this parameter as the optimality indicator. A good logistic regression model would predict a value closer to 1 for p_i when the field observation indicates that the stream is unstable (y_i=1), and a value closer to 0 when the stream is stable (y_i=0). According,

- When the stream is **unstable, y_i=1**, the second term becomes zero, and only the first term contributes to l. In this case, if p_i is **high** – which is **desirable**, $\ln[p_i]$ will be **a small negative number**, closer to 0; In contrary, if p_i is low – which is undesirable, $\ln[p_i]$ will be a large negative number.

- When the stream is **stable, y_i=0**, the first term becomes zero, and only the second term contributes to l. In this case, if p_i is high – which is undesirable, $\ln[1-p_i]$ will be a large negative number; In contrary, if p_i is **low** – which is **desirable**, $\ln[1-p_i]$ will be **a small negative number, closer to 0**.



Therefore, the best logistic regression model, associated with optimal b_o and b, results in the value of l closet to zero, i.e., being maximized. The log likelihood (L) may be alternately reported as the positive number G^2, -2.log likelihood, which is to be minimized to achieve the optimal b_o and b:

$$G^2 = -2 \cdot L$$

Hypothesis Testing

There are several types of hypothesis testing applicable to logistic regression. However, as only simple (one explanatory variable) logistic regression is being considered, only the test for overall significance is relevant. In this case, the null hypothesis is that the logistic regression model with the slope coefficient b is not significantly better than an intercept-only model (where b = 0, i.e., constant p). The statistic parameter used in this test is overall likelihood ratio (lr_o):

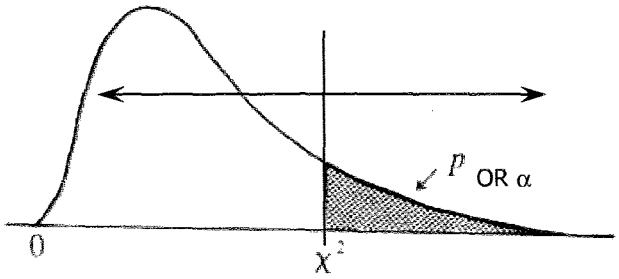
$$lr_o = (G_0^2 - G^2)$$

where G^2 is -2.log likelihood of the non-zero-b model and G_0^2 is that of the intercept-only model.

The overall likelihood lr_o can be approximated by a chi-square distribution with k degrees of freedom, where k is the number of slopes estimated (Helsel and Hirsch, 1992). In this case, k = 1. Consequently, if $lr_o > \chi^2_{1,\alpha}$, the null hypothesis can be rejected: b is not zero. In contrary, if the null hypothesis cannot be rejected, the best estimate over all values of the explanatory variable of p is simply the proportion of the data set that y = 1.

$|r_o| < \chi^2_{k,\alpha}$:

b does NOT differs from zero at a significance level of α;
G^2 is high;
I is low (a large negative number)



$|r_o| > \chi^2_{k,\alpha}$:

b differs from zero at a significance level of α;
G^2 is low;
I is high (close to zero)

Appendix B

APPENDIX B

TECHNICAL

MEMORANDUM #4

FROM: GeoSyntec Consultants

DATE: January 20, 2004

SUBJECT: **Evaluation of Range of Storms for HMP Performance Criteria**

Under Provision C.3.f.iv.3 of the SCVURPPP NPDES permit, "the HMP proposal shall identify the maximum rainfall event below which the standard applies, or range of rainfall events for which the standard applies"[1]. This memorandum provides a discussion of technical results for evaluating which flows, or range of flows, are the most important when considering stream channel erosion and hydromodification impacts. This analysis is based on the Erosion Potential (Ep) methodology developed on Thompson Creek. Results from the Ross Creek and San Tomas Creek studies will be incorporated into this work as they become available.

The results for two land development scenarios are being compared in this memo: pre-development represented by 1970 conditions and future development in 2020. The results and discussion should simply be considered as an example of a "before" and "after" watershed condition. The same analysis would be done if we were evaluating a proposed future development in a currently undeveloped watershed. We believe that it is important that the Work Group develops an understanding of this simple (and perhaps ideal) comparison before moving forward and applying the HMP to more complex mixed development scenarios (e.g., partial development, infill, etc.).

By making this comparison, there is no intent to suggest that cities should require developers to reduce existing runoff to pre-urban conditions. The proposed Standard (TM #3), for example, requires discharge from future development to be maintained at *existing conditions*. Whether the proposed development is located in an undeveloped watershed (or portion thereof) discharging to a healthy creek, or in a partially developed watershed discharging to an impacted creek, the intent is to not increase the current potential for erosion. Where the District has plans to repair and/or restore certain reaches of a stream, they would likely allow for known future development (refer to TM #3 for more specific details). In cases where a developer chooses not to participate with District's plans, or the development is unknown at the time the plans are made, the developer would be required to maintain *existing runoff conditions* and not increase the potential for erosion. In all cases, the HMP methodology can be used to measure changes in the erosion

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[1] RWQCB staff has indicated that, although paragraph C.3.f.iv.3 refers to rainfall events, management of hydromodification should be focused on runoff and stream flows (personal communication with Jan O'Hara, SFRWQCB). See discussion in Appendix A regarding the distinction between rainfall event and flow event frequencies or return periods.

potential, account for altered watershed and stream conditions, and evaluate proposed control measures.

This memo considers the case where the watershed is mostly undeveloped and is being evaluated for impacts caused by future build-out conditions. In this case, the baseline stream condition is assumed to be a healthy stable stream. Once the Work Group understands this scenario, the next step is to analyze the scenario where a development is proposed in an already developing watershed and the objective is to not increase the potential for erosion, i.e., maintain the existing conditions.

Section 1 presents the predicted range of geomorphically significant flows before development to show which flows appear most important in influencing sediment transport and the erosion processes. Section 2 presents similar results under developed conditions to show how this range of important flow changes. Section 3 presents a discussion on managing the impacts of hydromodification and which flows must be controlled to minimize impacts.

1 Geomorphically Significant Flows of Thompson Creek before Urban Development

Figure 1 presents the cumulative work curves predicted for a subset of stream cross sections in the Thompson Creek subwatershed when the subwatershed was relatively undeveloped.

The intent of Figure 1 is to show that the flows which appear most important in controlling sediment transport and the erosion processes range from near zero up to the 10-year peak flow. These curves illustrate that a significant amount of the total work done (approx. 90%) on the channel bed and banks is done by flows up to the 10-year peak flow. Flows up to the 2-year peak flow are predicted to perform from 30% to 60% of the total work done on the channel depending on the local flows, channel slope and geometry.

These results suggest that the appropriate range of flows to consider when managing hydromodification in this watershed is all flows up to the 10-year peak flow. This is consistent with the opinions of restoration professionals who have experience in the Bay Area, who believe that the channel forming discharge is greater than the 2-year event and can be as high as the 5- or 10-year event.

One important concept here is that no single storm event size is controlling the erosion processes and channel form, but that a wide range of storms up to the 10-year event are all influencing the physical characteristics of stream channels. The definition of geomorphically significant flows has been provided in previous HMP submittals.

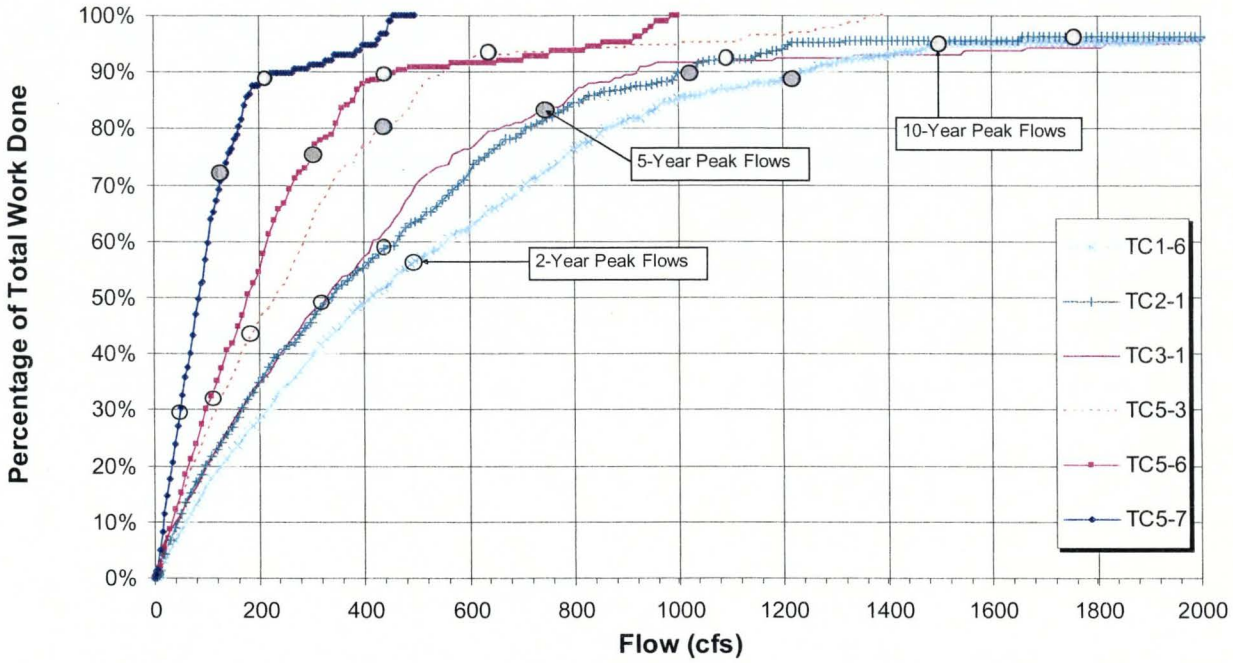


Figure 1 - Cumulative Work Curves Showing Range of Significant Flows before Development (using Thompson Creek stream segments as an example)

2 Geomorphically Significant Flows for the Future Built-Out Scenario

Figure 2 presents the cumulative work curves predicted for the same stream cross-sections after the Thompson Creek subwatershed is developed to the build-out condition, based on General Plan information. Comparison of this figure with Figure 1 shows the changes predicted in the cumulative work curves resulting from development of the subwatershed.

Generally, a larger percentage of work is done by lower flows than what was done before development. This is shown by the faster rise in the cumulative work curve between zero and approximately 300 cfs. The percentage of work done by flows up to the 2-year peak flow changes widely. The upper Segment 5 cross section (TC5-7) changes very little, while the others show work increases of 10% to 20% (e.g., for TC1-6, work done by flows up to the 2-year peak flow increases from 55% to 75%). The percentage of work done by flows up to the 10-year peak flow changes slightly.

As discussed above, these results help indicate which flows are contributing most to the total work done and suggest which flows should be managed for hydromodification. Flows less than and up to the 2-year event become more significant than they were before development, however, flows between the 2-year and the 10-year peak flow still contribute a significant percentage of the total. Management of hydromodification should definitely include flows up to the 2-year peak flow where most of the change is observed. However, flows greater than the 2-year peak flow are also causing increases in the total work done on the channel. This is discussed further in the following section.

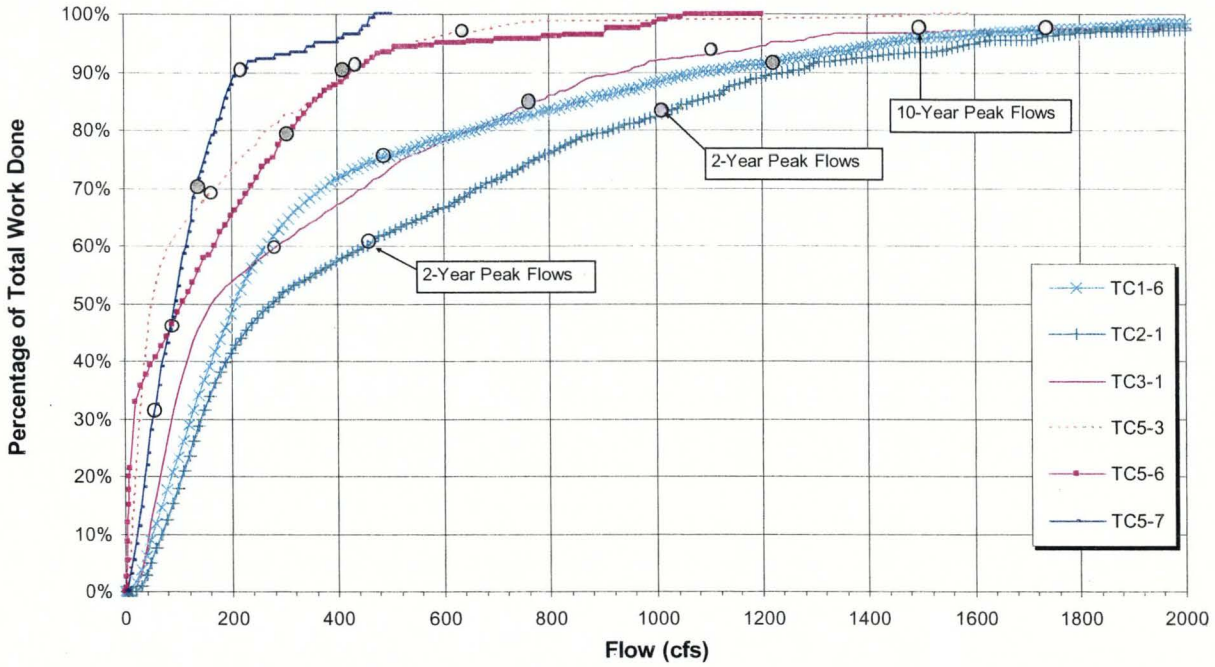


Figure 2 - Cumulative Work Curves Showing Range of Significant Flows after Development

3 Selecting the Range of Storms to Manage

Consider the case in which a developer wants to develop an existing undeveloped area. The developer chooses to implement the erosion potential option (see TM #3) and computes work curves such as those presented in Figure 3 at a cross section just downstream from his proposed discharge. The developer computes the cumulative work curve for the existing undeveloped land use condition (pre-development flows). The developer also computes the cumulative work curve for the proposed development without any flow controls in-place (post-development flows). The HMP performance criterion applied is to maintain the Ep ratio (the ratio of post-development to pre-development erosion potential) at 1.2 or less (i.e., within 20% of the predeveloped condition). In the example illustrated in Figure 3, the total work done for the existing undeveloped condition is 940 units. Allowing an increase of 20% (i.e., Ep ratio = 1.2) results in an allowable increase in work up to 1128 units. If flows are uncontrolled, the post-development Ep ratio would be 11 (10,340 work units) in this stream segment.

Equating the range of flows to manage to discrete storm sizes helps specify which flows should be managed to achieve the HMP objectives. Both the 2-year and 10-year peak flow magnitudes are shown on Figure 3.

There are two important flow ranges illustrated in Figure 3: 1) the range of flows to be managed, and 2) the flows that can be discharged uncontrolled (from an erosion perspective). The results indicate that flows must be controlled in the range where the post-development work curve increases most dramatically. Work must be matched up to a point where the remaining uncontrolled flows do not increase the total work beyond the allowable 1128 units. Although the

increase in work done by high flows is small relative to the increase caused by lower flows, the increase is still measurable. These higher uncontrolled flows add the allowable increase in work (20%).

Figures 4 and 5 show example post-development work curves for two other stream cross sections in Thompson Creek, each with smaller predicted Ep ratios for the post-development condition. Cross sections TC3-7 and TC1-6, with Ep ratios of 4 and 7 respectively, are presented to show the range of possible conditions and the variability in the extent of hydromodification impacts. When flow conditions are the same between cross sections, channel slope, critical shear stress and geometry determine the magnitude of the erosion potential.

All stream systems are likely to have reaches with varying degrees of predicted impacts depending on location specific channel characteristics, even within relatively short distances. Selecting management criteria on the basis of less sensitive reaches is only partially protective and could result in impacts to the more sensitive ones. This analysis suggests that, for Thompson Creek and similar watersheds on the eastern side of the Santa Clara Basin, all flows up to the 10-year peak flow should be controlled to fully protect streams from hydromodification caused by future development.

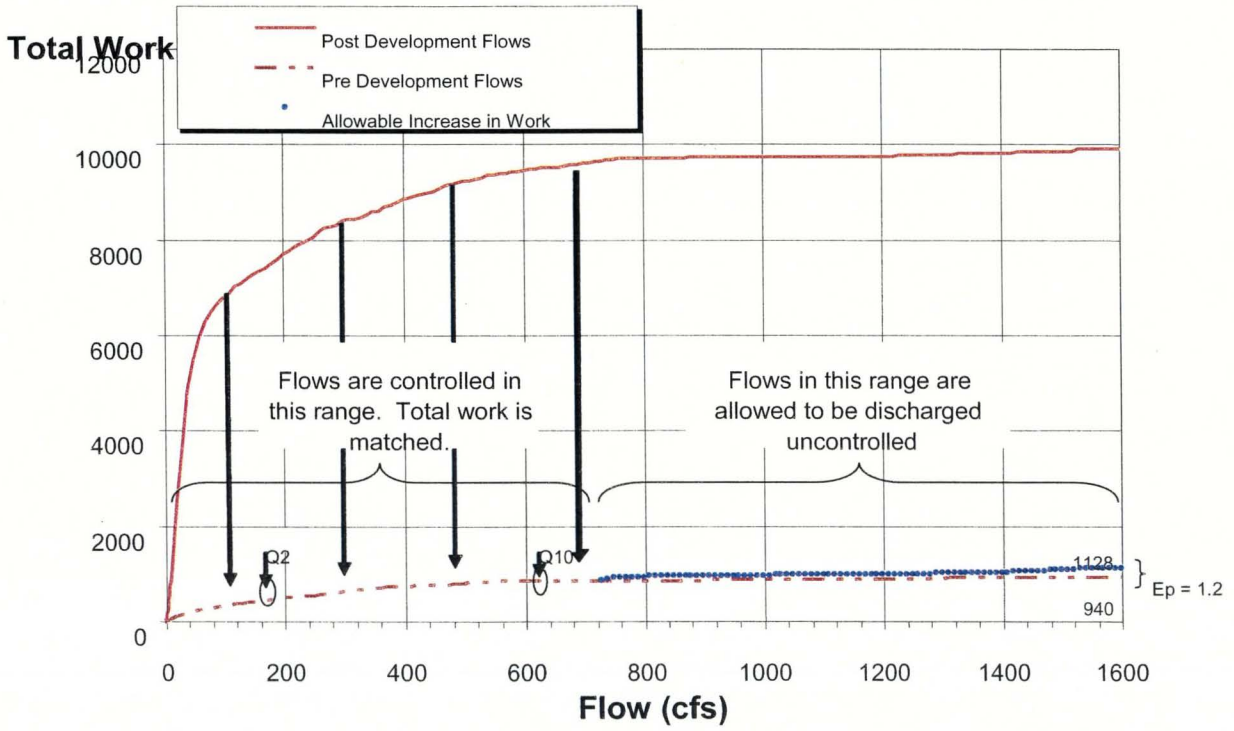


Figure 3 - Cumulative Work Curves Illustrating Range of Flows to Manage using Cross Section TC 5-4 (Ep ratio for uncontrolled post-development condition = 11)

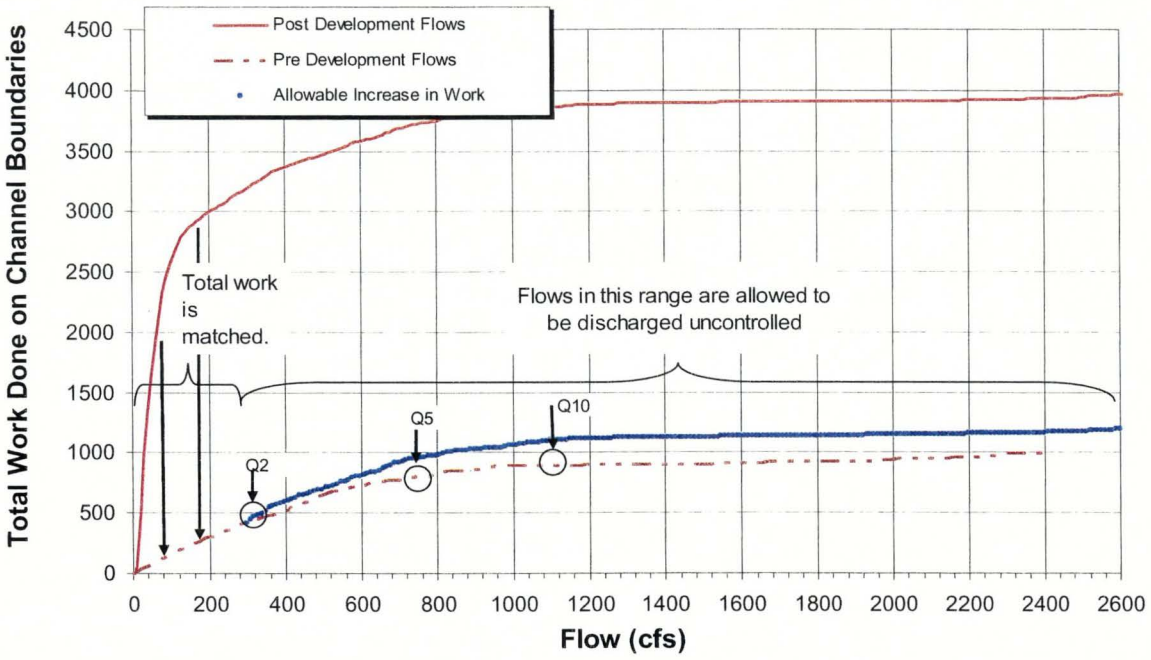


Figure 4 - Cumulative Work Curves for Cross Section TC3-7 (Ep ratio for uncontrolled post-development condition = 4)

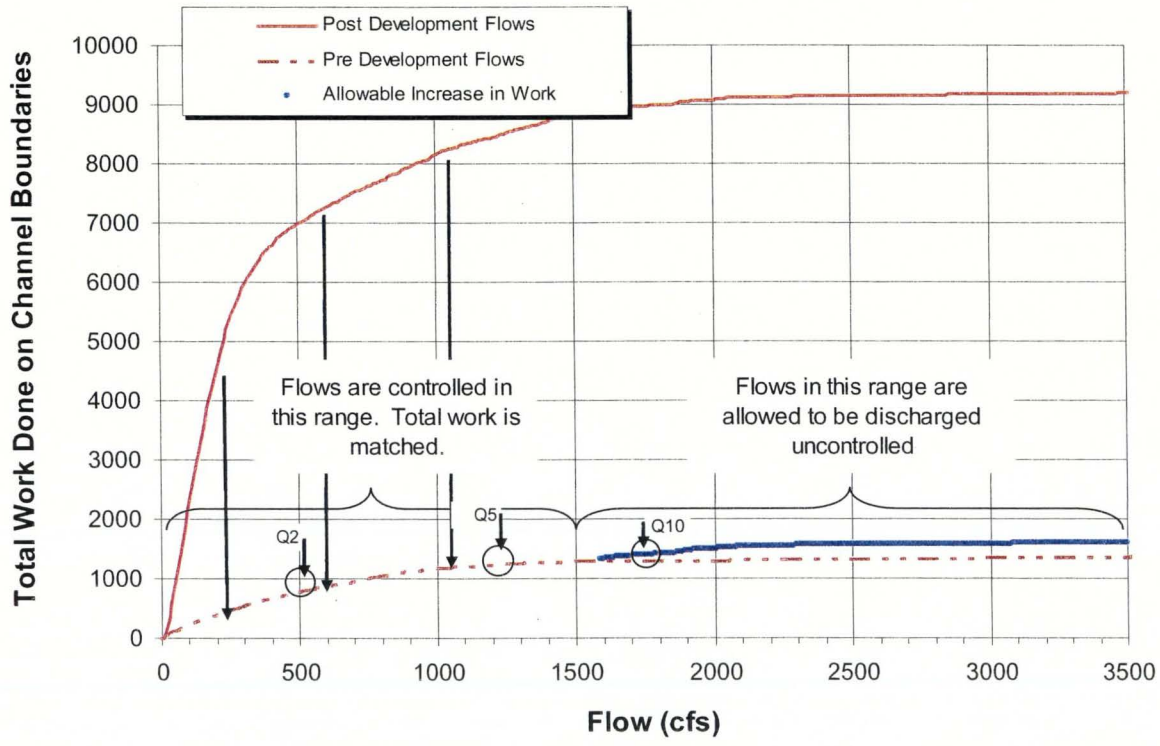


Figure 5 - Cumulative Work Curves for Cross Section TC1-6 (Ep ratio for uncontrolled post-development condition = 7)

3.1 Discussion on the Range of Storms to Manage

There have been some questions raised regarding whether selecting the 10-year peak flow as the upper limit for the range of storms to manage is conservatively high, and whether this selection is appropriate Basin wide. This section provides a response to these questions.

Within the Thompson Creek study area, measured Ep ratios vary due to stream channel characteristics, with medium and high ratios intermixed with low ratios. There are several factors for this, but slope is an easy one to consider. Figure 6 provides a simplified illustration of a longitudinal profile with both shallow slopes and steep slopes downstream from an outfall location.

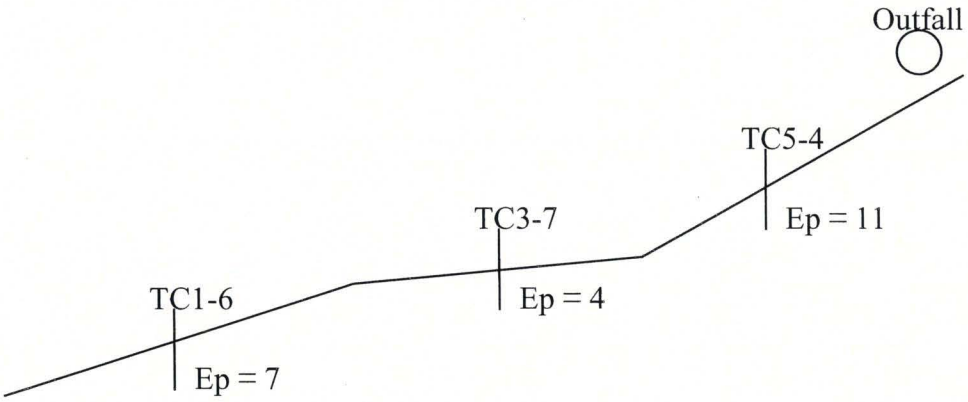


Figure 6 – Illustration of Longitudinal Profile Showing Cross Sections and Outfall

1. Considering this illustration and the results discussed above, a range of flows should be selected to be fully protective of all stream segments downstream of the discharge point. For this reason, the 10-year peak flow is believed to be an appropriate upper limit for the range of flows to be managed within the Thompson Creek Subwatershed.
2. Both flow duration and volume control basins are sized to control the *difference* in runoff volume between pre- and post- condition storm events, not the entire event volume. Results show that the difference in runoff volume between pre- and post- 10-year storm events is roughly equivalent to the total 1.1-year, or slightly larger, storm event volume.
3. Western Washington selected the 50-year peak flow as its upper limit. Our tests show flows greater than the 10-year peak flows contribute less than 5% to 10% of the total work done on the stream channel.

The assumption that Thompson Creek is representative of other east side subwatersheds was one of the basic assumptions made at the beginning of the project when the Lower Silver/Thompson Creek Subwatershed was selected as the first test watershed. The results from the Thompson Creek assessment, indicating that the climate, soils, geology, and vegetation characteristics of this subwatershed are the same as those for the east side watersheds, confirms this assumption.

For this reason, the 10-year peak flow is believed to be an appropriate upper limit for other east side subwatersheds of the Coyote Watershed, as well as Thompson Creek.

Extrapolating these results to the west side watersheds is much less certain, which is one of the reasons why the Ross/San Tomas subwatersheds were selected as additional test subwatersheds. Results from these subwatersheds will help us determine if aggregating results is appropriate, or whether different criteria must be developed.

T E C H N I C A L

M E M O R A N D U M # 5

FROM: GeoSyntec Consultants

DATE: January 26, 2004

SUBJECT: **Evaluation of Volume Control Effectiveness**

Background

This Technical Memorandum (TM) #5 provides a discussion on the effectiveness of using volume control to manage hydromodification, for review and discussion by the OMM Subgroup and the HMP Work Group.

TM #3 discusses a proposed hydromodification standard, performance criteria, and implementation options. The three performance strategies that can be used include: project runoff volume control, project runoff flow duration control, and in-stream erosion potential (Ep) control. These strategies differ by the level of detail in the analysis and costs for implementation. Although the erosion potential strategy is state-of-the-art and considered the most accurate approach to evaluate HMP impacts and control measures, the project team recognized that a simpler approach such as volume control may be needed for smaller projects.

Volume control means that only the amount of runoff generated from the existing (pre-project) site may be discharged from the site after development, and the difference between the post-project and pre-project volumes must be retained on-site or discharged to a non-eroding stream segment. The performance criteria for the volume control strategy does not stipulate the shape of the post-project discharge hydrograph, only that its volume (i.e., the area under the hydrograph) must be the same as the pre-project conditions. However, there is concern that this approach may not be as accurate as flow duration control in maintaining the existing Ep of stream segment receiving the discharge. This memorandum evaluates and discusses the accuracy of volume control by comparing the work done on the stream by different shaped hydrographs using the erosion potential concept.

Methodology

The evaluation was performed using the discrete events as defined in the District's Design Flood Flows Manual ("Green Book", 1979) and modeled for the Thompson Creek subwatershed. The method used was to compare the work done by various post-development hydrographs with volume control to the pre-project hydrograph (in this case, the "pre-urban" hydrograph for segments of Thompson Creek), which is assumed to represent the target work value.

The questions to be addressed are:

1) What is the affect of hydrograph shape on in-stream work when storm runoff volume is maintained at pre-project levels?

2) Is volume control effective?

To evaluate the effect of hydrograph shape while maintaining pre-project volume, the work done by several hydrographs of different shapes is compared to the work done by the pre-project hydrograph. It was recognized in TM #3 that volume control is not as accurate as flow duration control because it does not account for the differences in erosive power for the same excess volume at higher flows as compared to lower flows. Referring back to the work index in the draft HMP (equation 1, Section 5.3, GeoSyntec 2002), the reader will notice that work is a non-linear function of excess shear stress. This non-linearity results in these differences in erosive power between hydrographs of the same volume which have different flow rates.

The test conducted was to compare the amount of work done on a stream channel between hydrographs of different shapes, but having the exact same total volume (area under hydrograph). Post-project hydrographs were selected to represent a range of possible conditions between the post-project hydrograph and the pre-project hydrograph (Figure 1). It is assumed that a developer would begin with a post-project hydrograph and then consider BMPs to mitigate the effects. Uniform hydrographs were selected to have peaks ranging from the post-project peak to the pre-project peak, with duration changing as needed to maintain pre-project runoff volume. Tests 1 through 3 were selected in this manner (Figure 1). Test 4 was selected by assuming a volume control BMP captured the beginning portion of the storm until it is filled, at which time the remaining runoff is discharged uncontrolled. Test 4 consists of the remaining portion of the existing conditions hydrograph that is not captured in the BMP, and has a volume equal to the pre-project volume as required.

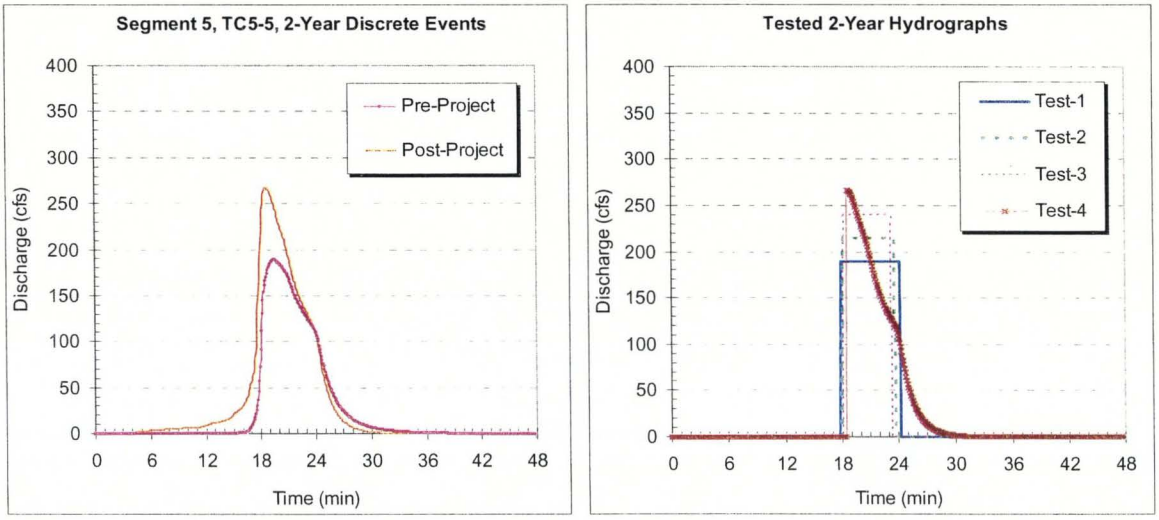


Figure 1. Example of Hydrographs Used in Analysis

The work done by each of these hydrographs was computed and compared. The results are summarized in Table 1 below.

Following the example described above, hydrographs were developed for the 1.1-year, 2-year, 5-year, and 10-year discrete events, for Segment 1, Segment 5, and for a small sub-catchment discharging to Misery Creek, a tributary to Thompson Creek. Two cross sections per segment were used in the evaluation. For example, in Segment 5, both TC5-4 and TC5-6 were evaluated and each of these cross sections had 4 tests conducted, so all total there were 8 tests conducted per segment. The results for Segment 5 and Misery Creek are plotted in Figures 2 and 3.

Discussion

Table 1 shows that each of the tested hydrographs does more work on the channel than the pre-project 2-year storm event. With channel geometry and discharge volume held constant, the differences in work are solely a result of changing the hydrograph shapes.

Table 1. Estimated Work Done at TC5-4 by 2-Year Hydrographs of Varying Shape

Hydrographs	Peak Flow (cfs)	WORK (ft-lbs/sq.ft.)	Difference in Work from Pre-Project (ft-lbs/sq.ft.)	Percent Higher than Pre-Project Work
Pre-Project	189	**555,937**		
Post-Project	267	**763,129**	+207192	37.3
Test 1	189	656,481	+100544	18.1
Test 2	215	681,109	+125171	22.5
Test 3	240	703,608	+147671	26.6
Test 4	266	615,000	+59063	10.6

Other hydrographs, such as uniform hydrographs with smaller peak flows than pre-project and triangular shaped hydrographs, could be selected to bring these differences closer to the pre-project value. However, there are some observations that we can use to infer the results of most other shapes.

With volume remaining constant, the following conditions are observed:

- Higher peaked hydrographs do more work than lower peaked hydrographs.
- Test 1 has the same peak as the pre-project hydrograph, but also does more work than the pre-project hydrograph because of concentrating volume in a shorter duration.
- Test 4 produces the least difference in work when compared to pre-project, because flows are distributed over a longer time with a portion of the hydrograph volume discharged at low peak rates creating the long tail of the hydrograph.

Figure 2 plots the results for Segment 5 for all the design storms tested. The results express the range of potential work done in excess of the pre-project amount. For example, the results for the 2-year storm show a range of work from 1 to 32 percent greater than the pre-project amount, depending on the hydrograph shape. The range for 1.1-year events is significantly higher at 300

percent (this large increase in work would be created by a uniform hydrograph with a high peak flow). The minimum value of 1 is assumed because it is possible to find shapes that create work very close to the pre-project amount.

It is interesting that the magnitude of work done above the pre-project amount gets smaller as the storm size increases. The reason this occurs is not clear, but, it appears that the difference between pre and post conditions is larger, percentage wise, for smaller volume hydrographs than larger ones.

This trend is also observed when comparing downstream cross sections to upstream cross sections. What this is suggesting is that smaller catchments discharging to smaller creeks could potentially have higher unexpected increases in work and stream channel erosion, than what is shown in Figure 2.

To test this theory, a volume analysis was conducted on a 250 acre sub-catchment discharging to Misery Creek a tributary to Thompson Creek. The previous cross sections have contributing catchment areas of 2,400 acres or more. This example is more similar to the applicable size range stipulated in draft performance criteria (TM #3).

As expected, the tests of this smaller catchment area show higher increases in work above the targeted pre-project amount. For example, work done by a post 2-year hydrograph is estimated to be up to 80% more than the pre-project amount. For both 5 and 10-year storms, the work done by the post-project runoff hydrographs could be up to 50% more than the pre-project hydrograph.

This analysis suggests that specifying only volume control could lead to unexpected increases in work and stream erosion, unless other criteria are specified that control the shape of potential discharge hydrographs. One approach would be to require "hydrograph matching" instead of volume matching. Hydrograph matching maintains the volume and distribution of flows for a single discrete storm event in a similar manner as that used for flow duration control.

Conclusions

Conclusions that can be drawn from the volume control evaluation include the following:

- Hydrograph shape does matter and unexpected stream erosion could occur when managing volume control alone.
- Volume control does not appear effective unless other controls are imposed to maintain the pre-project hydrograph shape.
- Adding peak flow matching to the performance criteria helps improve effectiveness, but is still not enough. For example, Test 1 in Figure 1 has the same peak as the pre-project hydrograph, yet it does more work because the tails of the pre-project hydrograph are concentrated in a shorted duration uniform hydrograph.
- The volume control performance criteria in TM #3 may need to be changed to specify "hydrograph" matching, as opposed to "volume" matching.

Next Steps

GeoSyntec is currently developing tests using the flow duration matching strategy. A draft TM #7 will be distributed soon for the Work Group to review. GeoSyntec plans to use the flow duration example to test the effectiveness of hydrograph matching.

For example, a BMP sized for discrete event hydrograph matching will be completed. Then this basin will be tested as to how well it replicates the pre-project flow duration curve. This will be completed for a basin sized to match the 2-, 5-, and 10-year discrete event hydrographs.

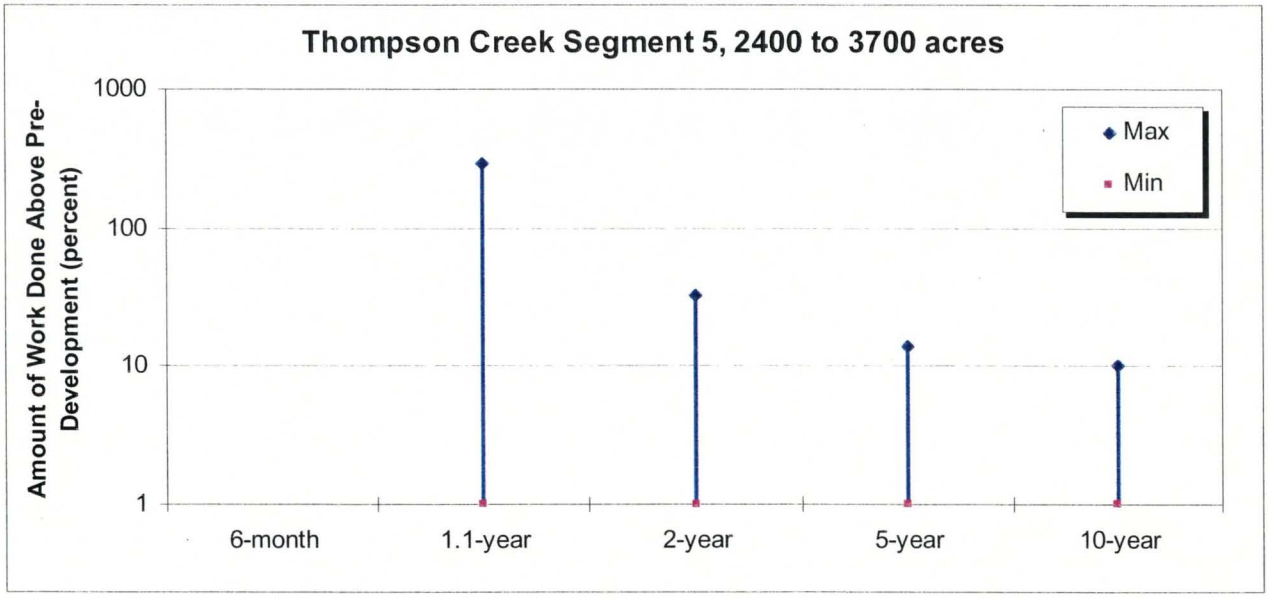


Figure 2. Difference in Work Done for Varying Hydrograph Shapes Having the Same Volume.

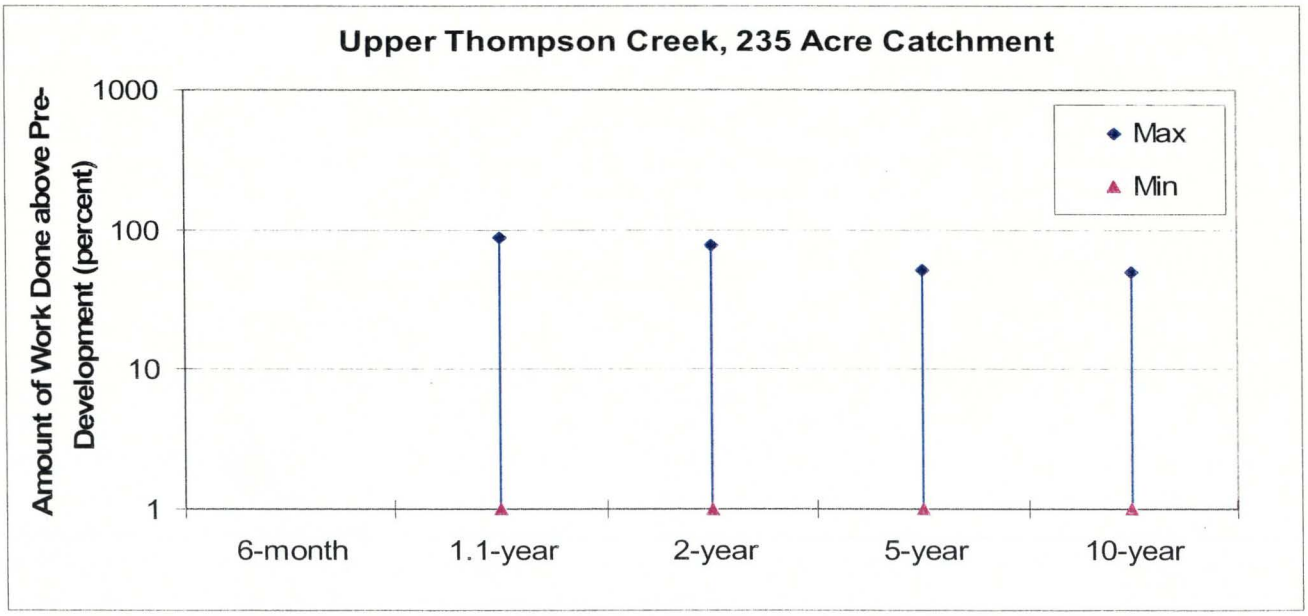


Figure 3. Difference in Work Done for Varying Hydrograph Shapes for a 250 acre Catchment.

TECHNICAL

MEMORANDUM # 7

FROM: GeoSyntec Consultants

DATE: February 3, 2004

SUBJECT: **Flow Duration Control Example**

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1 Introduction

Technical Memorandum #3 – Draft Hydromodification Control Standard, Performance Criteria, and Implementation Options, includes a hydromodification management strategy centered on maintaining the pre-project flow duration curves. Matching flow duration of runoff from project areas is an effective way to manage hydromodification and maintain the in-stream erosion potential.

The draft standard requires that the pre-project flow duration curves be maintained for all flows between a lower limit and an upper limit (The HMP OMM Subgroup is currently evaluating the appropriate range of storms.). Flow duration control is similar to volume control but is more accurate for managing hydromodification than volume control alone. Hydrograph matching improves on volume control, but is for a single discrete event as opposed to the full record of flows. Matching flow duration curves maintain runoff volume for the full distribution of flows from the lower and upper limit. Unlike volume control or hydrograph matching, flow duration control considers the full multi-year discharge record, including antecedent conditions and back-to-back storms. The objective of this analysis is to apply the flow duration control strategy as described in TM#3 (Section 3.2.3). This memo discusses the application of this strategy on a 716-acre residential development in Thompson Creek subwatershed (Figure 1) and for a 274-acre development in southern California (Figure 2). The flow duration results of these two projects are compared.

Flow duration control requires that the increase is surface runoff created by the installation of impervious surfaces be retained on-site and discharged at less than the critical flow of the stream, Qc (see TM #3). However, Qc is assumed to be zero in this example. The developer does not need to know information about the stream using this assumption. As a result, the BMPs considered must be able to capture and store a design volume and then dispose of this volume through infiltration and/or evapotranspiration.

The type of facility considered and evaluated is a surface basin including detention and outlet control, storage and infiltration. Infiltration is assumed to take place within the flow duration basin, as opposed to a separate infiltration facility. By-passes and low-impact design (LID)

strategies that minimize the change in surface runoff should be considered, although this example is focused on sizing and costs of the flow duration basin without LID strategies.

2 Flow Duration Basin Configuration

The flow duration approach involves routing a time series (50 years of record in this case) of continuous runoff from the future development through a detention basin that diverts and retains a certain portion of the runoff. This portion to be retained is essentially the increase in surface runoff volume created between a pre-project and post-project condition. This captured runoff is assumed to be infiltrated in the basin for this example.

The flow duration basin is designed to have two pools (Figure 3), a low flow pool (Zone A) and a high flow pool (Zone B). The low flow pool is designed to capture small to moderate size storms, the initial portions of larger storms, and dry weather flows. The high flow pool is designed to store and release higher flows to maintain, to the extent possible, the pre-project runoff conditions. The flow duration basin also serves as a water quality treatment facility and can be designed to treat dry and wet weather flows using a combination of extended detention and natural treatment processes. Most dry weather "nuisance flows" will also infiltrate in the basin.

The flow duration basin is sized using an iterative process of adjusting basin storage as well as selecting and adjusting orifice sizes in the outlet structure. The low flow pool within the basin is initially sized to capture the increase in runoff volume that is generated from the impervious surfaces. This capture volume is dependent on the development characteristics, the soil types, and the magnitude of change in runoff created by the proposed development. Previous analyses have shown that area requirements have less to do with the range of storms selected for management and more to do with site and development characteristics.

Once the lower pool is sized to capture the correct volume of runoff, the upper pool of the basin is sized to detain and discharge larger flows through a specific set of orifices in such a way as to reproduce the flow duration curve. The number, diameter, and elevation of these orifices are determined by a trial and error approach first developed in Western Washington (King County, 1998). The combination of sizing the lower portion of the basin and the upper portion to detain and discharge high flows has the affect of capturing the correct volume of runoff and matching the pre-development distribution of hourly flows.

In this example, the basin functions as an infiltration basin such that the excess runoff volume is infiltrated. Separate flow duration-infiltration basins may be needed for water quality purposes, or a by-pass pipeline could carry the excess runoff to a safe discharge location. Combining flow duration and infiltration into a single facility reduces the overall land requirements for stormwater management.

The outlet structure is designed to reproduce the pre-developed flow duration (runoff histogram) using orifice diameter and elevation above the bottom of the basin. Figure 4 illustrates the outlet structure. The number, size and placement will vary from basin to basin depending on project conditions. The headwall should be constructed using steel plates in such a manner that allows owner/operators to change the outlet configuration to improve basin performance if necessary.

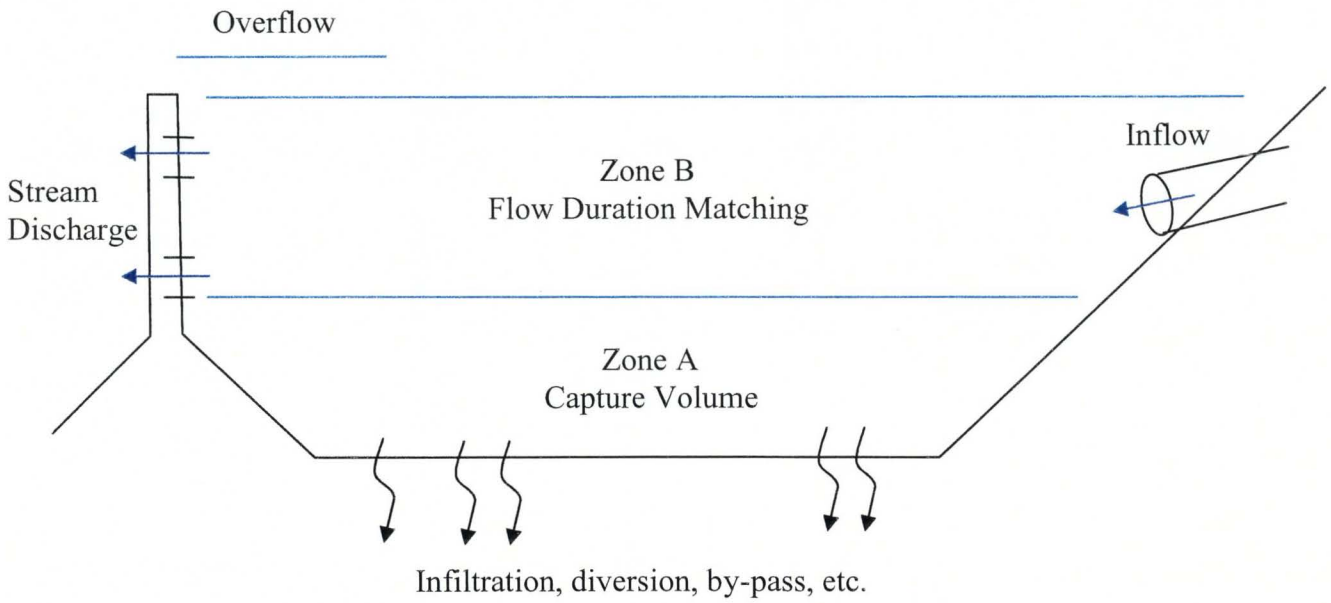


Figure 3. Generalized Configuration of Flow Duration Basin

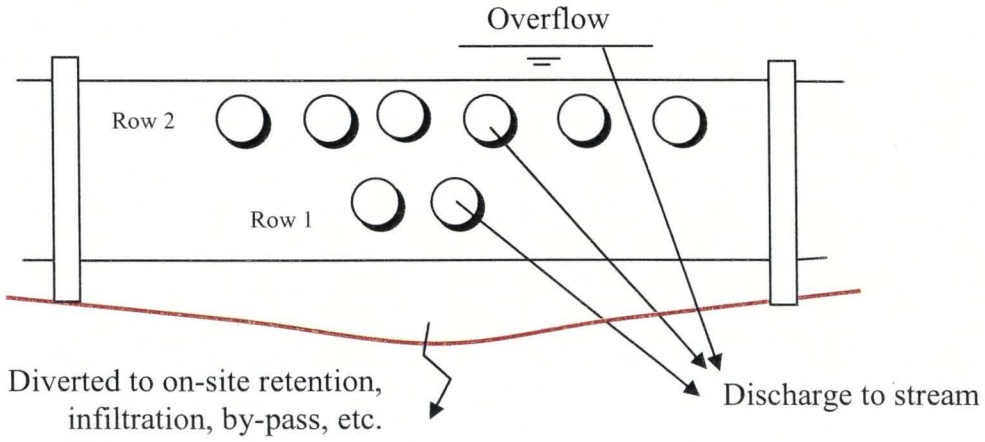


Figure 4. Generalized Configuration of Outlet Structure

3 Thompson Creek Example

The Lower Silver – Thompson Creek subwatershed (approximately 42 square miles) is located within the Coyote Creek watershed, the largest watershed in the Santa Clara Basin, with some of its area partially in the City of San Jose and partially in unincorporated Santa Clara County.

Thompson Creek originates in the Diablo Range foothills at an elevation of about 2,300 feet and presently flows northerly to its confluence with Lower Silver Creek near the Eastridge Shopping Center at an elevation of approximately 125 feet. Its tributary streams include, from north to south: Quimby Creek, Fowler Creek, Evergreen Creek, Yerba Buena Creek, and Cribari Creek. Thompson Creek discharges to Lower Silver Creek. The Thompson Creek subwatershed encompasses about 17.5 square miles.

This example sizes HMP controls for an existing 716-acre residential development between Yerba Buena Creek and upper Thompson Creek, as if HMP controls were required when the project was constructed (i.e., the pre-project condition is the "pre-urban" condition). This area has 71% impervious cover. Existing land use is predominantly single family residential, streets, and park and golf course properties. The infiltration rate for soils at the basin site is assumed to be 0.2 inches/hour.

3.1 Results

This section summarizes the flow duration sizing results for the Thompson Creek example.

Figure 5 presents the results of this method illustrating the pre-project flow duration curve, post-project (future) and post-project curves with controls. The post-development curve illustrates that the effect of development is to increase the duration of flows; that is, the flow duration curve moves to the right indicating that both volume and duration of flows increase. Also note that this is a logarithmic scale on the horizontal axis, so small changes along the axis may indicate large changes in volume and duration. The effect of the BMP is to reduce the durations to more closely replicate the existing condition. This means that the magnitude of hourly runoff and the number of hours (duration) that flows occur at those magnitudes are nearly the same between pre- and post-project conditions.

For the 716 acres of development, the resulting basin characteristics are as follows:

- Basin volume is 120 acre-feet.
- Basin surface area is 15 acres (2% of the sub-catchment area), assuming a maximum basin depth of 10-feet and 3:1 side slopes..
- Fifteen orifices having diameters of 9-inches each discharge to the creek.

When comparing the pre-project curve to the post-project with BMP curve, the results shown in Figure 5 would be considered a very good match. Design of flow duration basins is somewhat of a trial and error process, and some design configurations match pre-project curves better than others. Western Washington allows no more than a 10% deviation between the curves over no more than 10% of the curves' length, or range of flows.

The shape of the post-project with BMP curve is dependent on the type of outlet structure. In our case, we are using orifice holes in a headwall, following Western Washington's guidelines. Orifice holes create the convex shaped sections of the curve, where each section represents the change in flows from adding the next row of orifice holes. In the example in Figure 5, there are three convex curve sections between zero and 51 cfs. Above 51 cfs, flows are spilling over the overflow structure.

3.1.1 Cost Curves

Figure 6 shows the construction cost curves for a surface basin infiltration facility. This curve was derived from well detailed cost spreadsheets, including such elements as site preparation, earth work, structures, piping, re-vegetation and so forth. Detailed spreadsheets are available for review. Land costs of $1 Million per acre are included in the final estimate.

The cost estimates presented in this section represent a planning level cost estimate, and represent an accuracy of + 50% to -30%.

The Thompson Creek example construction costs would be $1.6 Million just for the basin itself. This would not include pipelines, design, environmental documents, or land costs. Assuming $1 Million per acre of land costs and 15 acres needed, then $15 Million is required for land.

The $1.6 Million construction cost is equivalent to $2,250 per acre of development area. Assuming 4 houses per acre in this location, this flow duration basin construction costs $563 per lot. Adding land costs for a total of $16.6 Million, works out to be $5,800 per lot.

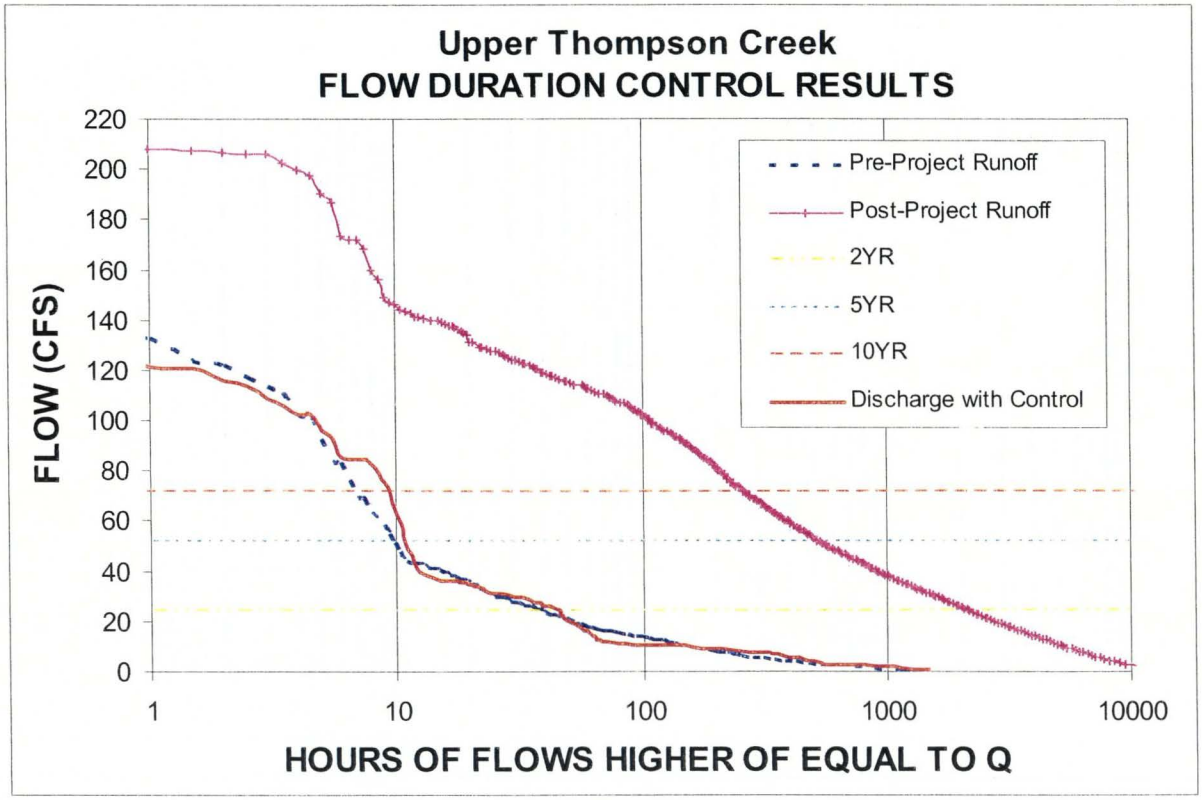


Figure 5. Flow Duration Curves for Thompson Creek Example

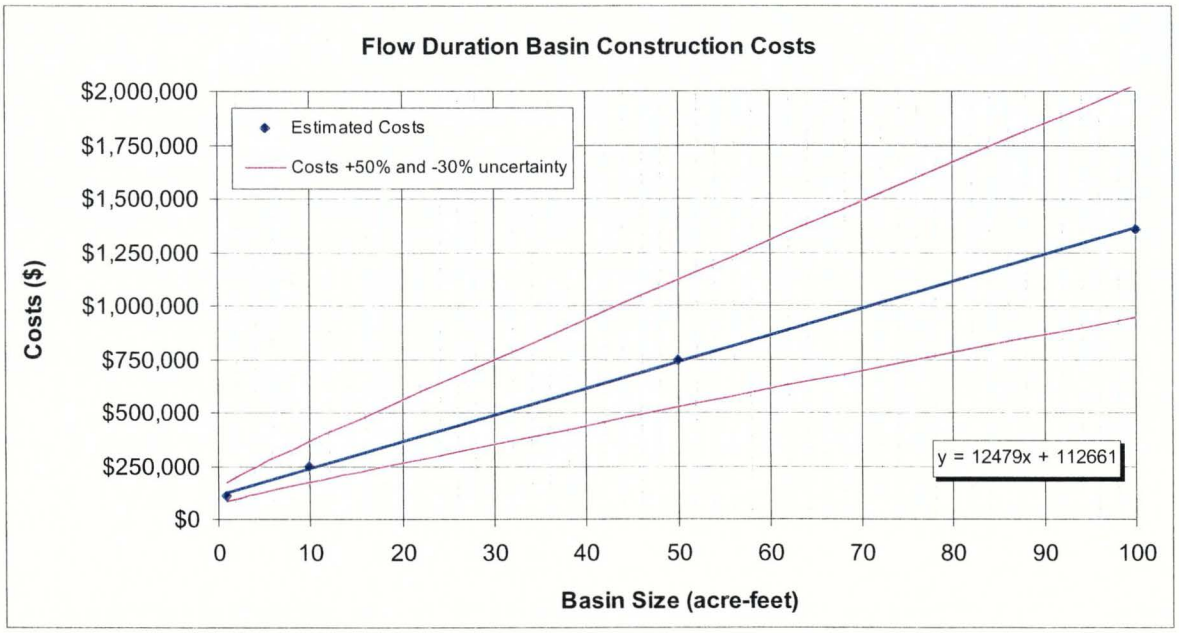


Figure 6. Estimated Cost for Surface Basin

4 Discussion of Gobernadora Creek (Orange County) Example

GeoSyntec is responsible for preparing the Water Quality Management Plan (WQMP) for a project in Southern California to support planning efforts of the San Juan and Western San Mateo Watersheds. The purpose of the WQMP is to assess potential effects of the proposed project on water quality, water balance, and hydromodification and recommend control measures to address potential impacts. The Southern California project consists of nine sub-watersheds evaluated separately, one of which is Gobernadora Creek subbasin.

One of the objectives of the this project is to strategically layout the development to protect natural resources and reserve certain areas, like high infiltration soils, for stormwater management facilities. The results discussed in this section show the effect of strategic site planning when compared to the Thompson Creek example. The reader will see that the difference between the pre- and post-project flow duration curves is small when compared to Thompson Creek example.

The Gobernadora subbasin is an elongated valley that is characterized by deep alluvial deposits in the canyon floor. Hill slopes and ridges are overlain by hardpan or exposed rock. The proposed development addresses approximately 2,156 acres. Approximately 881 acres would remain as open space, with the remainder being developed into estate, single, and multi-family residential housing, golf resort, and transportation.

The Gobernadora subbasin has ten sub-catchments. The flow duration basin sizing results for Sub-catchment 3 is discussed here. Sub-catchment 3 consists of 274 acres of single family residential area and transportation corridors, at 44 percent imperviousness. The pervious area is mostly grassland and riparian area.

4.1 Results

Figure 7 presents the results for a Gobernadora Creek example. The post-project flow duration curve with BMPs closely matches the pre-project curve, with better accuracy than the curves shown in Figure 5. This means that the magnitude of hourly runoff and the number of hours (duration) that flows occur at those magnitudes are nearly the same between pre- and post-project conditions.

For the 274 acres of development, the resulting basin characteristics are as follows:

- Basin volume is 15 acre-feet.
- Basin surface area is 3.7 acres (or 1.4% of the sub-catchment area) assuming maximum basin depth of 5 feet and 3:1 side slopes.
- 5 orifices having diameters of 7 to 20-inches each discharge to the creek.

Of all the flow duration basins sized for Gobernadora, the range of land area required for the facilities ranged from 1.2 to 1.4 percent of their total sub-catchment areas. It should be noted that the soil infiltration rate where the flow duration basins are located were assumed to have 1 inch/hour infiltration rates. The range of percent imperviousness is 40 to 44%. One sub-catchment has 50% imperviousness with development on sandier soils. The resulting basin is 2.8% of its catchment area.

When comparing Figures 5 and 7, notice that the difference between the pre- and post-project conditions for development in Gobernadora Creek watershed is much smaller than the differences observed in Thompson Creek subwatershed. In Gobernadora Creek watershed, residential development was located on low infiltration clay soil and bedrock, reserving the high infiltration soils for stormwater management BMP's.

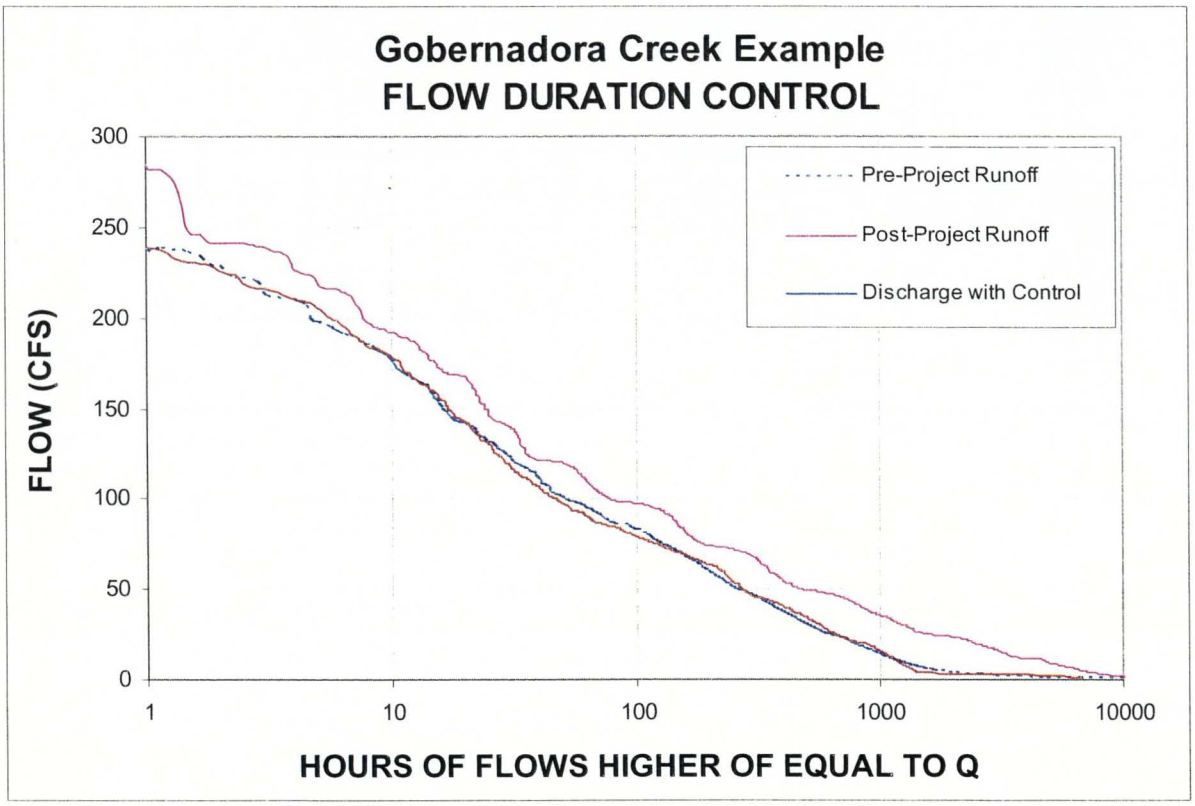


Figure 7. Flow Duration Curves for Gobernadora Creek Example

5 Hydrograph Matching (Volume Control) Effectiveness

In TM #5 – Volume Control, volume control as a strategy was found not to be very effective at controlling hydromodification impacts, because of the differences in erosive power for the same volume at higher flows when compared to lower flows. The memo concluded that hydrograph matching may significantly improve the results by not only controlling runoff volume, but also controlling the shape of the hydrograph and preventing higher flows from occurring.

This section presents and discusses an effectiveness evaluation of hydrograph matching when compared to the full 50-year flow duration matching option using the 716-acre sub-catchment in Thompson Creek. Matching the full flow duration curve is the most effective option at maintaining the pre-development erosion potential.

This analysis compares the effectiveness of different sizing strategies at reproducing the pre-development flow duration curve. Four control basins were sized and their outlets designed using three independent discrete storm sizes (2-year, 10-year, and 50-year), and one sized using the full 50-year hour record of runoff generated by the HEC-HMS hydrologic model. The basins sized according to the discrete events were sized and designed to match the discrete hydrograph, such that both pre-project volume and shape are maintained.

Figure 7a, 7b and 7c present the results of matching the discrete event hydrographs. Both the storm hydrographs and flow duration curves are plotted for each storm size. In all cases, the storm hydrographs were matched closely in volume and shape. The basin discharge hydrograph

lags the pre-project hydrograph by a couple of hours, but this is not a problem. The result for matching the full flow duration curve was presented in Figure 4 above.

Note that storm volume is also matched with this approach and a portion of the basin captures and retains the difference in volume between the pre- and post-project hydrographs. This capture volume is slowly siphoned off to infiltration or some other disposal facility.

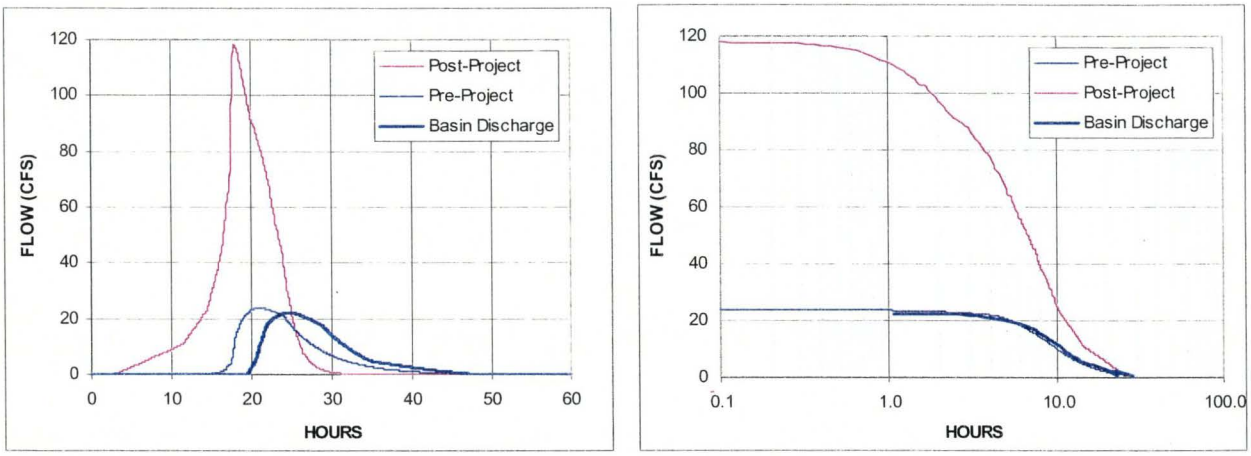


Figure 7a. 2-Year Discrete Hydrograph Matching Results

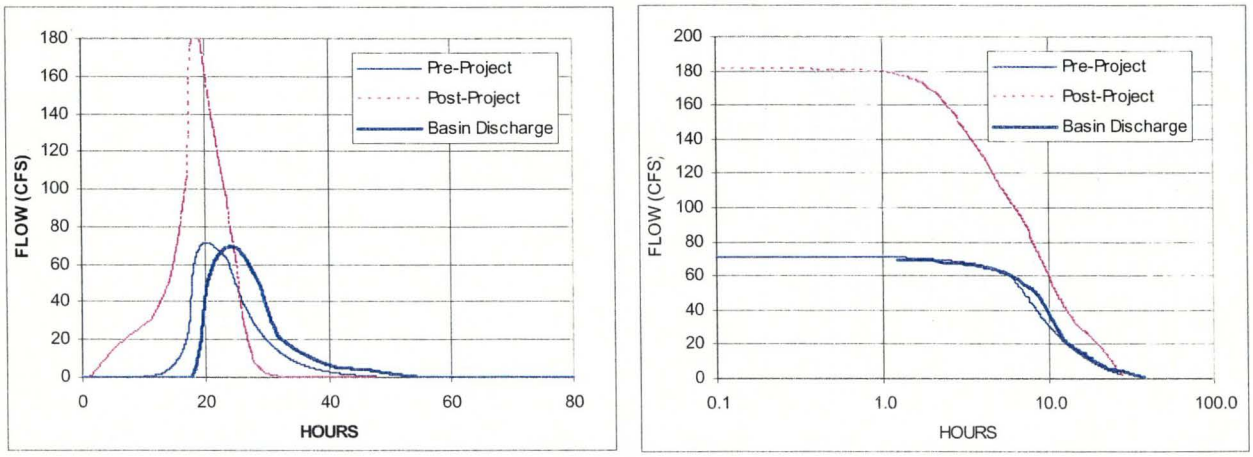


Figure 7b. 10-Year Discrete Hydrograph Matching Results

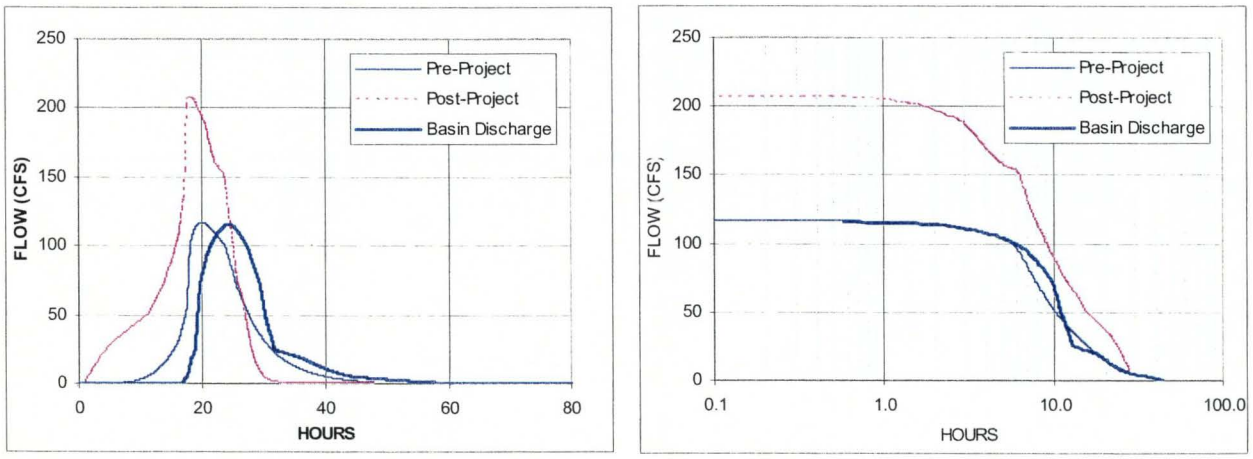


Figure 7c. 50-Year Discrete Hydrograph Matching Results

In TM #3 – Draft Standard, one of the control options is intended to provide a simple discrete event based approach that could replicate the control effectiveness of continuous simulation analysis and flow duration matching. The next test that was conducted was to evaluate the effectiveness of the discrete basins by routing the full 50-year flow record through them and plotting the resulting flow duration curves.

Figure 8 below plots the resulting flow duration curves for each of the four sizing criteria, 2-year, 10-year, 50-year, and the full 50-year time series. As shown, the 50-year time series approach closely reproduces the pre-project flow duration curve. The three basins sized using discrete events do not match the pre-project curve and would result in stream channel erosion if a large percentage of the watershed development was designed using discrete events.

Also shown in Figure 8 are the resulting basin sizes. Basin sizes range from 71 acre-feet to 155 acre-feet for the 2-year and 50-year storms, respectively. Note that the size of the 50-year time series basin (116 acre-feet) is close to the basin size using the 10-year storm (113 acre-feet). Also recall that these basins are being sized to capture the difference in runoff volume between pre- and post-project conditions, not the entire storm event.

Generally, the end points of the flow duration curves reflect the different basin sizes. For example at the low flow end, the 2-year basin releases more small flows to the stream than the 10-year of 50-year basins, all of which release about 5 to 10 times more than the pre-project condition. The high flow end shows the affect of large storm peak flows. The 2-year basin is small such that large storms spill over the overflow uncontrolled. The 10-year and 50-year basins can provide peak control of the larger events and closely approximate pre-urban peaks. The middle portion of the flow duration curves is primarily influenced by the outlet structure. The shape of the discrete basin curves reflect the design of the outlet structure, where each break in the curve shows the affect of stormwater discharge through orifice holes at higher elevations.

One interesting observation is that the 50-year discrete basin does not match the full time series curve even though its basin size is larger. The reason for this is in the design of the outlet structure, which was designed to reproduce the 50-year hydrograph alone.

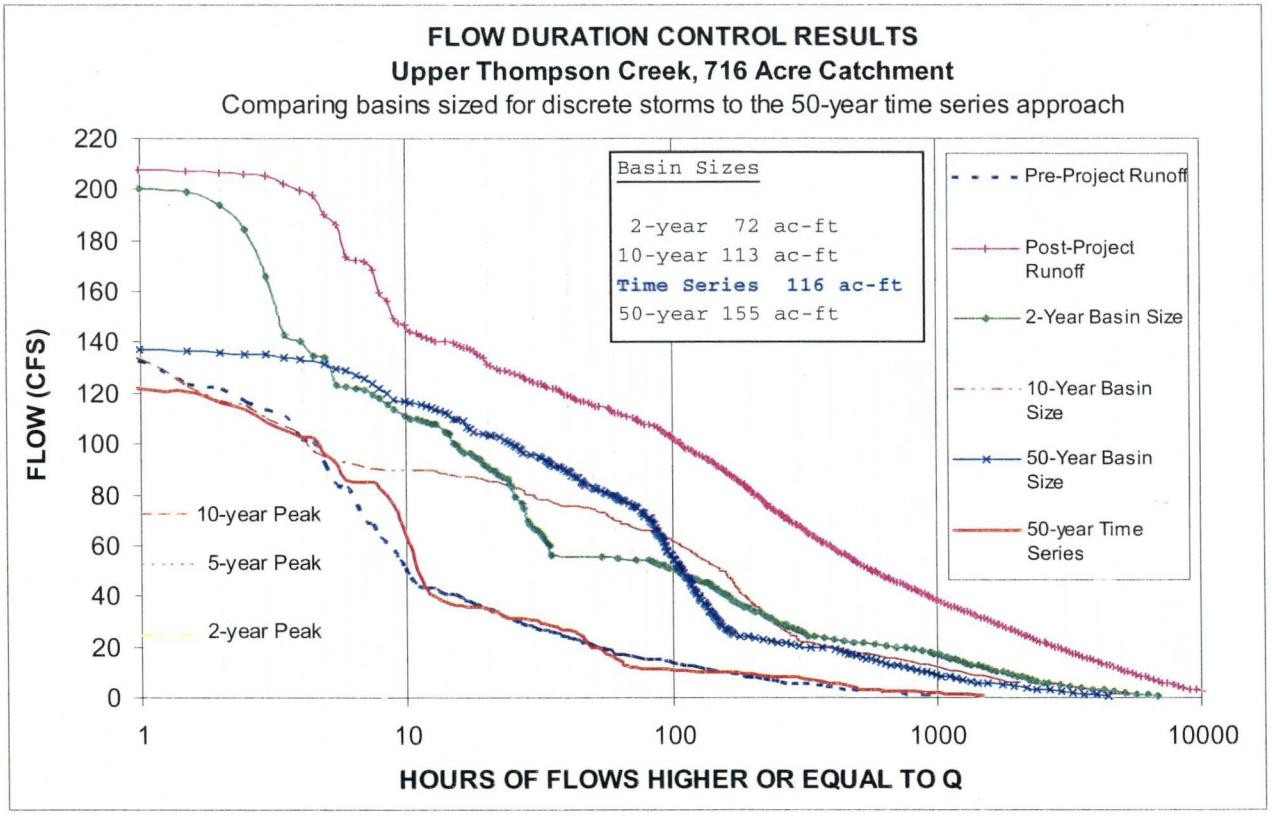


Figure 8. Comparison of Different Basin Sizing Criteria

6 Conclusions

Sizing hydromodification control basins using discrete events does not closely reproduce the pre-project flow-duration conditions and will lead to stream channel erosion if widely used throughout a catchment.

The required area for flow duration basins seems to be between 1 to 3 percent of the contributing catchment area. Where projects with good soil infiltration rates and low-impact development strategies require less land area (1.2 to 1.4 percent). Areas where soil infiltration rate is low and the percent of imperviousness high, land requirements could be 2 to 3 percent.

If infiltration cannot be achieved in the flow duration basin and pre-treatment is required before infiltration, than a separate infiltration facility may be required. In this case, additional land area would be required. Base on the examples completed by GeoSyntec, about twice as much land area would be required if flow duration and infiltration had to be done in separate facilities.